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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Volume 25

September 1947-February 1950

U. S. DEPARTMENT OF COMMERCE

BUREAU OF PUBLIC ROADS

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Triaxial Compression Test Results Applied to Flexible Pavement Design

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by E. S. BARBER, Highway Engineer

This discussion concerns several relations between triaxial compression test results and the thickness of flexible pavement based on stress distribution in elastic materials and the strength of plastic materials. Tables and graphs of stresses, displacements, and bearing capacity under normal and tangential surface loads are presented. Their application to the design of thickness of flexible pavement courses is outlined, using unit values for the components derived from triaxial compression tests. Designs based on allowable displacement, overstress at a point, and total failure are considered. The analyses are not proposed as determinative design procedures but are rather intended as aids in studying the behavior of pavements in service, and as guides in planning and conducting experimental studies of the interaction of pavements and subgrades.

THE SELECTION of thickness of flexible pavement (surface and base) required to support a given loading over a given subgrade is generally based on service records and observations of field performance of similar pavements constructed on similar subgrades. To evaluate the similarity of various pavement materials and subgrades, a method of testing the materials is necessary. In correlating observations of pavement performance involving various loads and materials, it is helpful to have a theoretical relation between the pavement thickness and the material test results.

Thus, while the results of field observations are required, their application may be broadened and their evaluation made more quantitative if they are correlated by means of test results on the component materials with the aid of theoretical relations. The following discussion concerns several relations between triaxial compression test results and pavement thickness based on stress distribution in elastic materials and the strength of plastic materials.[1]

ELASTIC STRESS DISTRIBUTION

The earth may be considered as an elastic mass, bounded only by the surface horizontal plane and hence semi-infinite in extent. When such a mass is subjected at the surface to a uniform vertical pressure over a circular area, every element in the mass is acted upon by the stresses shown in figure 1. Three planes are considered to intersect at the element: The horizontal plane, parallel to the surface; the radial plane, vertical and along the radial line between the load center and the element; and the tangential plane, vertical and perpendicular to the radial plane. Acting upon the element, in these three planes, are the normal stresses shown in figure 1: The tangential normal stress, p_t; the radial normal stress, p_r; and the vertical normal stress, p_z. Two equal shear stresses are developed: The vertical shear stress, s_z, acting in the vertical direction on the tangential plane; and the radial shear stress, s_r, acting in the radial direction on the horizontal plane. The tangential normal stress is a principal stress, and therefore there are no shear stresses in the plane on which it acts.

These stresses have been evaluated analytically by Love (*1*)[2] and tabulated numerically by Tufts (*2*). Some corrections were made to Tufts' tabulation by means of influence charts

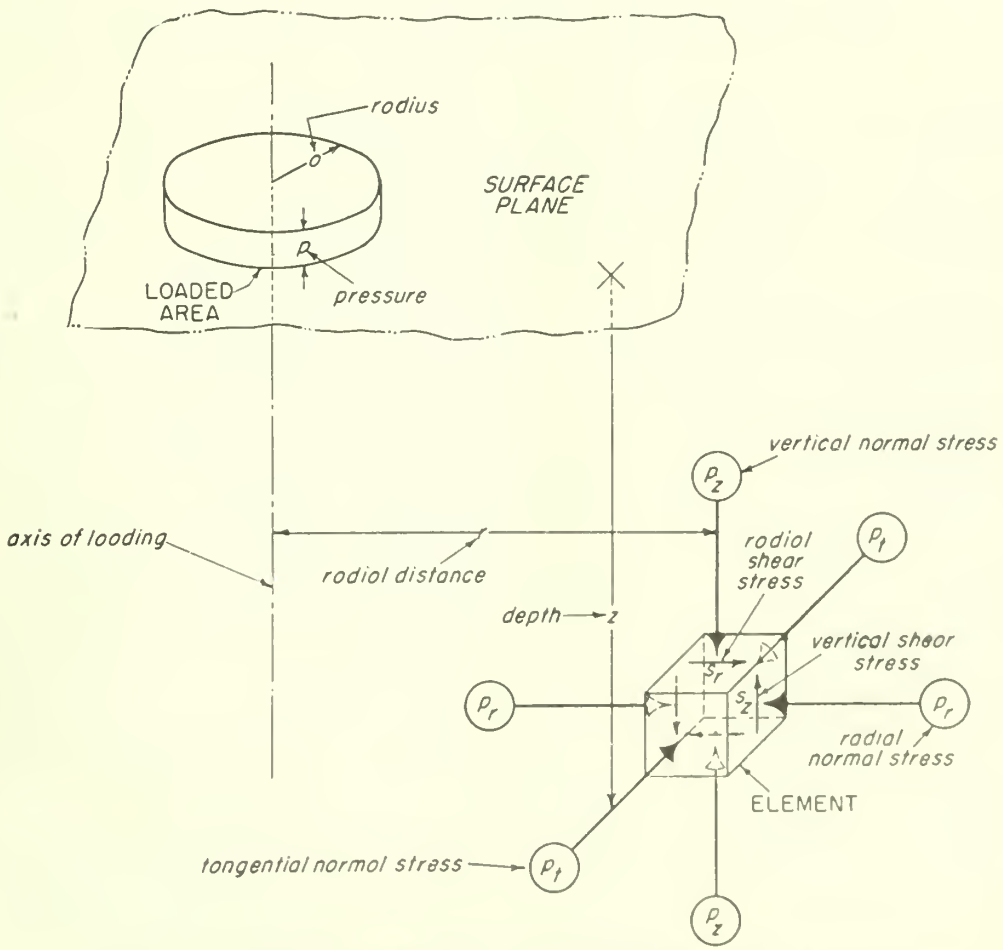

Figure 1.—Stresses on an element from pressure applied at the surface.

[1] A complete notation for this article appears on page 8.
[2] Italic numbers in parentheses refer to the bibliography, p. 8.

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Table 1.—*Ratio of stresses transmitted to a point in a semi-infinite mass from a surface load uniformly distributed over a circular area*

VERTICAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_z}{p}$

Depth of point÷radius, $\frac{z}{a}$	Horizontal radial distance÷radius, $\frac{r}{a}$								
	0	0.25	0.5	1.0	1.5	2.0	2.5	3.0	4.0
0.25	0.986	0.983	0.964	0.460	0.015	0.002	0.000	0.000	0.000
0.5	.911	.895	.840	.418	.060	.010	.003	.000	.000
0.75	.784	.762	.691	.374	.105	.025	.010	.002	.000
1.0	.646	.625	.560	.335	.125	.043	.016	.007	.000
1.25	.524	.508	.455	.295	.135	.057	.023	.010	.001
1.5	.424	.413	.374	.256	.137	.064	.029	.013	.002
1.75	.346	.336	.309	.223	.135	.071	.037	.018	.004
2.0	.284	.277	.258	.194	.127	.073	.041	.022	.006
2.5	.200	.196	.186	.150	.109	.073	.044	.028	.011
3	.146	.143	.137	.117	.091	.066	.045	.031	.015
4	.087	.086	.083	.076	.061	.052	.041	.031	.018
5	.057	.057	.056	.052	.045	.039	.033	.027	.018

RADIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_r}{p}$ (POISSON'S RATIO=0.5)

0.25	0.643	0.626	0.565	0.385	0.144	0.058	0.028	0.014	0.004
0.5	.374	.360	.325	.286	.196	.098	.050	.027	.008
0.75	.208	.204	.196	.209	.175	.112	.064	.044	.012
1.0	.116	.118	.123	.149	.146	.104	.069	.045	.022
1.25	.067	.072	.080	.107	.116	.096	.069	.047	.026
1.5	.040	.046	.055	.078	.091	.082	.064	.047	.026
1.75	.025	.028	.035	.056	.070	.068	.058	.046	.027
2.0	.016	.019	.024	.041	.053	.057	.052	.042	.027
2.5	.008	.009	.013	.023	.033	.038	.038	.035	.025
3	.004	.006	.008	.014	.021	.026	.028	.026	.022
4	.001	.002	.003	.006	.009	.012	.015	.016	.016
5	.001	.001	.002	.003	.005	.007	.008	.009	.010

TANGENTIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_t}{p}$ (POISSON'S RATIO=0.5)

0.25	0.643	0.628	0.580	0.243	0.019	0.005	0.001	0.000	0.000
0.5	.374	.359	.317	.141	.028	.007	.003	.001	.000
0.75	.208	.197	.170	.085	.025	.008	.003	.001	.000
1.0	.116	.109	.096	.054	.021	.008	.003	.001	.000
1.25	.067	.063	.056	.035	.016	.007	.003	.001	.000
1.5	.040	.037	.034	.023	.012	.006	.003	.001	.000
1.75	.025	.024	.022	.015	.009	.005	.003	.001	.000
2.0	.016	.015	.014	.011	.007	.004	.002	.001	.000
2.5	.008	.007	.007	.006	.004	.003	.002	.001	.000
3	.004	.004	.003	.003	.002	.002	.001	.001	.000
4	.001	.001	.001	.001	.001	.001	.000	.000	.000
5	.001	.001	.000	.000	.000	.000	.000	.000	.000

RADIAL SHEAR STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{s_r}{p}$

0.25	0.000	0.024	0.065	0.299	0.042	0.014	0.003	0.002	0.001
0.5	.000	.057	.129	.262	.102	.032	.013	.006	.002
0.75	.000	.069	.141	.221	.128	.053	.024	.013	.003
1.0	.000	.065	.124	.178	.128	.069	.033	.018	.007
1.25	.000	.053	.101	.146	.118	.072	.039	.023	.010
1.5	.000	.041	.080	.119	.104	.071	.045	.028	.012
1.75	.000	.033	.062	.094	.091	.068	.046	.030	.014
2.0	.000	.026	.048	.070	.078	.062	.045	.032	.015
2.5	.000	.016	.030	.050	.056	.050	.041	.032	.018
3	.000	.009	.019	.034	.040	.040	.035	.029	.018
4	.000	.005	.009	.018	.022	.024	.024	.022	.016
5	.000	.002	.005	.010	.013	.015	.016	.016	.013

RADIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_r}{p}$ (POISSON'S RATIO=0)

0.25	0.265	0.248	0.199	0.082	[1] −0.026	−0.045	−0.061	−0.046	−0.026
0.5	.098	.087	.063	.075	−.056	−.006	−.016	−.018	−.017
0.75	.008	.008	.009	.055	−.067	−.033	−.009	−.006	−.010
1.0	−.030	−.025	−.015	.033	−.058	−.041	−.021	−.008	.000
1.25	−.043	−.036	−.024	.018	−.044	−.040	−.026	−.015	.004
1.5	−.044	−.038	−.028	−.007	−.032	−.035	−.027	−.019	−.006
1.75	−.041	−.036	−.028	−.001	−.022	−.028	−.025	−.019	−.009
2.0	−.037	−.033	−.024	−.006	−.013	−.022	−.022	−.019	−.010
2.5	−.028	−.026	−.022	−.010	−.004	−.012	−.016	−.016	−.011
3	−.022	−.020	−.017	−.010	−.002	−.006	−.010	−.011	−.010
4	−.014	−.012	−.011	−.009	−.004	−.001	−.003	−.005	−.007
5	−.009	−.009	−.008	−.007	−.004	−.002	−.000	−.001	−.004

TANGENTIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_t}{p}$ (POISSON'S RATIO=0)

0.25	0.265	0.259	0.245	0.183	0.132	0.095	0.061	0.046	0.026
0.5	.098	.094	.085	.070	.074	.061	.048	.037	.022
0.75	.008	.007	.007	.017	.034	.038	.034	.029	.019
1.0	−.030	−.030	−.025	−.009	.011	.020	.023	.021	.016
1.25	−.043	−.042	−.037	−.021	−.002	.009	.014	.015	.013
1.5	−.044	−.043	−.039	−.025	−.010	.001	.007	.010	.010
1.75	−.041	−.040	−.037	−.026	−.013	−.003	.003	.006	.008
2.0	−.037	−.036	−.033	−.025	−.015	−.007	−.001	.003	.006
2.5	−.028	−.028	−.026	−.022	−.015	−.010	−.004	−.001	.002
3	−.022	−.021	−.020	−.018	−.014	−.010	−.006	−.004	.000
4	−.014	−.014	−.014	−.012	−.010	−.008	−.006	−.005	−.002
5	−.009	−.009	−.009	−.009	−.008	−.007	−.006	−.005	−.002

[1] Minus sign indicates tensile stress.

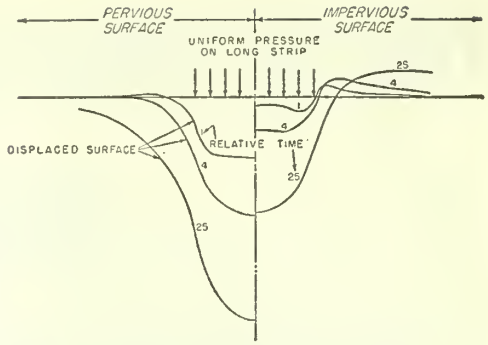


Figure 2.—Effect of time of consolidation on displacement of pervious and impervious loaded surfaces.

(*3*) to derive table 1, which shows the ratio of these stresses to the pressure applied at the surface, for various depths and for various radial distances measured from the center of the loaded area in terms of its radius. The vertical normal stress and the horizontal radial shear stress are independent of Poisson's ratio (the ratio of lateral to axial strain in a simple compression test). The radial and tangential normal stresses are given for Poisson's ratio of 0 and 0.5. Stresses for Poisson's ratio other than 0 and 0.5 may be obtained by direct interpolation: thus, the stress for Poisson's ratio of 0.25 is halfway between the tabulated values.

Integration of the stresses shown in table 1 gives the vertical displacement factors shown in table 2. While the stresses in a homogeneous mass are independent of the modulus of elasticity, the displacement is inversely proportional to the modulus. The displacement may be divided into two parts—that due to volume change only, corresponding to Poisson's ratio=0; and that due to lateral displacement at constant volume, corresponding to Poisson's ratio=0.5.

In comparing displacements measured in field tests with calculated values, consideration must be given to the fact that displacement due to volume change of wet soils is delayed because of the time required for the stress to be transferred to the soil solids as part of the water is forced out. This time effect and its dependence upon the permeability of the surface (*4*) is illustrated in figure 2. The curves

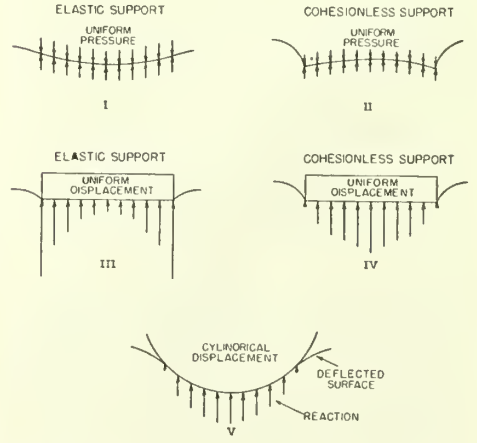


Figure 3.—Reactions under surface loads.

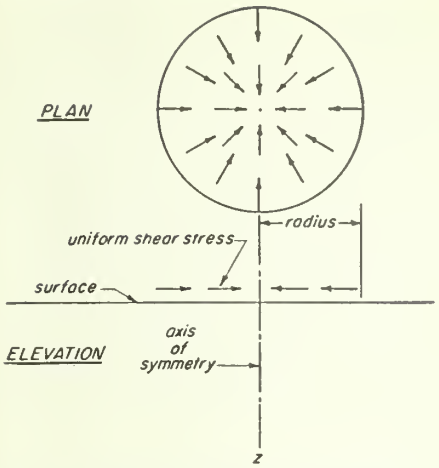


Figure 4.—Inward acting shear stresses produced by pneumatic tires.

on the left represent settlements when the surface is pervious, those on the right show the settlements at the same time when no drainage can occur at the surface. The relation between stress, volume change, and time for soil samples is determined in the consolidation test whereby a disk of soil is encircled with a metal ring and compressed between two pervious plates.

The time effect must also be considered when viscous materials are loaded. Under constant pressure, the surface displacement outside the loaded area may increase at first and then decrease as the displacement under the load continues to increase (*5*).

Figure 3, case I, shows the deflection of the surface of an elastic mass caused by a uniform pressure. If the supporting material is overstressed by a uniform pressure, the edge deflection increases and may exceed that at the center as indicated in figure 3, case II, for cohesionless support. If an elastic circular bearing block is placed between a uniform pressure and an elastic support, the reaction is concentrated toward the edge of the block (*6*). As the rigidity of the block increases, the edge stress increases without limit giving a uniform displacement (fig. 3, case III) equal to either 0.923 times the average displacement under a uniform pressure or 0.785 times the axial displacement under a uniform pressure (fig. 3, case I). Because of the lack of bearing capacity of unconfined cohesionless material, a rigid block on the surface of sand produces a reaction concentrated on the axis of loading (fig. 3, case IV).

The reaction under smooth-faced rollers and wheels without pneumatic tires approaches that shown in figure 3, case V, under a rigid solid cylinder. The average reaction is 0.785 times the maximum reaction (*7*).

The maximum reaction is

$$\sqrt{\frac{P'E}{R(1-\mu^2)}}$$

where: P' = load per unit length,
R = radius of loading cylinder,
E = modulus of elasticity of the elastic support,
μ = Poisson's ratio of the elastic support.

The reaction thus increases with increases in the load and modulus of elasticity, E, and decreases with increases in the radius, R.

Another factor which must be considered in evaluating loading tests is the production of inward acting shear stresses at the surface under pneumatic tires (*8*) as shown in figure 4. While measured shear stresses show a maximum at the quarter points of the diameter of the loaded area, their effect may be approximated by the vertical normal stress and displacement shown in table 3 for a uniformly applied shear stress (*9*). These normal stresses added to those due to the vertical applied pressures could account for the fact that Spangler (*10*) measured pressures transmitted to a strong base through a surface course which were greater than the inflation pressure of the tire used for applying the load. The shear stress also tends to increase the axial displacement more than that at the edge.

Table 2.—*Factors for calculation of displacement due to uniform pressure over a circular area* [1]

DISPLACEMENT FACTOR FOR POISSON'S RATIO=0.5

Depth of point÷radius, $\frac{z}{a}$	Horizontal radial distance÷radius, $\frac{r}{a}$										
	0	0.25	0.5	0.75	1.0	1.25	1.5	2	2.5	3	4
0	1.50	1.48	1.40	1.25	0.95	0.66	0.54	0.39	0.30	0.26	0.18
0.5	1.34	1.31	1.23	1.09	.89	.68	.55	.40	.31	.27	.19
1	1.06	1.05	.98	.89	.78	.66	.56	.41	.32	.27	.19
1.5	.83	.83	.79	.73	.67	.60	.52	.40	.32	.28	.20
2	.67	.67	.65	.62	.57	.53	.48	.39	.32	.28	.20
3	.47	.47	.46	.45	.43	.41	.38	.34	.30	.27	.21
4	.36	.36	.35	.35	.34	.33	.32	.30	.28	.25	.20
5	.29	.29	.29	.29	.29	.28	.28	.25	.24	.23	.19

DISPLACEMENT FACTOR FOR POISSON'S RATIO=0

0	2.00	1.97	1.86	1.67	1.27	0.88	0.71	0.52	0.41	0.34	0.25
0.5	1.51	1.49	1.39	1.24	1.06	.85	.68	.51	.41	.34	.25
1	1.12	1.11	1.04	.94	.85	.74	.65	.50	.40	.34	.25
1.5	.86	.85	.81	.76	.70	.65	.59	.47	.39	.34	.25
2	.68	.67	.65	.62	.59	.55	.52	.44	.38	.33	.25
3	.48	.47	.46	.45	.44	.42	.40	.37	.33	.30	.24
4	.37	.36	.35	.35	.35	.34	.33	.31	.29	.26	.22
5	.29	.29	.29	.29	.29	.28	.28	.26	.25	.24	.20

[1] Displacement, $D=\frac{\text{pressure}\times\text{radius}}{\text{modulus of elasticity}}\times\text{displacement factor}=\frac{pa}{E}F$; hence displacement factor, $F=\frac{DE}{pa}$.

Table 3.—*Axial vertical stress and displacement under uniform shear stress applied over circular area*

Depth of point ÷ radius	Vertical normal stress ÷ applied stress	Displacement factor [1] (Poisson's ratio=0)
0	1.00	1.00
0.25	.91	.76
0.5	.72	.55
1	.35	.29
1.5	.17	.17
2	.09	.11
3	.03	.05
5	.01	.02

[1] Displacement $=\frac{\text{shear stress}\times\text{radius}}{\text{modulus of elasticity}}\times\text{factor}$.

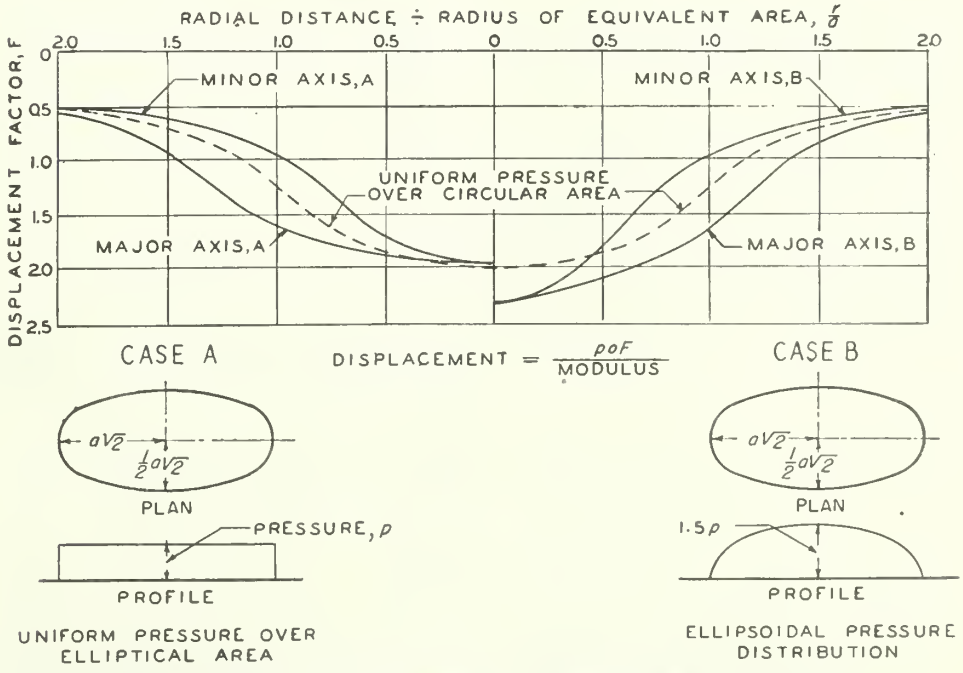


Figure 5.—Surface displacements for various pressure distributions.

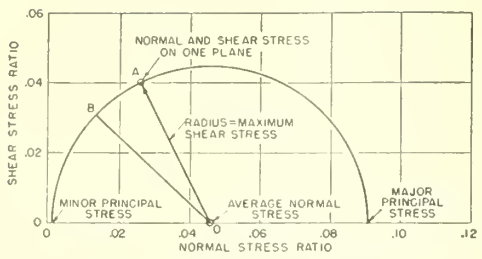


Figure 6.—Stresses at a point.

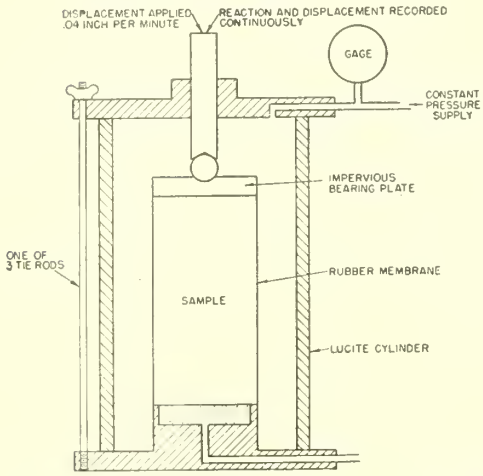


Figure 7.—Essentials of the triaxial compression test.

The theoretical surface displacements for three distributions of vertically applied pressure (*11*) are shown in figure 5. A uniform pressure over an elliptical area slightly smaller (about 10 percent) than the contact area (fig. 5, case A) is a better approximation of measured vertical pressures under pneumatic tires (*12*) than an ellipsoidal pressure distribution (fig. 5, case B). While the displacement is different on the major and minor axes, the average displacement is very close to that for a uniform pressure over a circular area.

In the following discussion, the load, P, is considered as the inflation pressure of the loaded tire, p, applied vertically and uniformly distributed over a circular area of radius, a, giving the relation $P=p\pi a^2$.

STRESSES AT A POINT

The stresses shown in figure 1 and table 1 are those acting at a point in the vertical and horizontal directions. The stresses at a point in other directions can be calculated from those given in table 1. Thus, for a point with coordinates $\frac{z}{a}=3$ and $\frac{r}{a}=2$ with Poisson's ratio of 0.5, table 1 gives $\frac{p_z}{p}=0.066$, $\frac{p_r}{p}=0.026$, and $\frac{s_r}{p}=0.040$. The average normal stress is the average of the normal stresses in any two mutually perpendicular directions, which for this point is $\frac{0.066+0.026}{2}=0.046$.

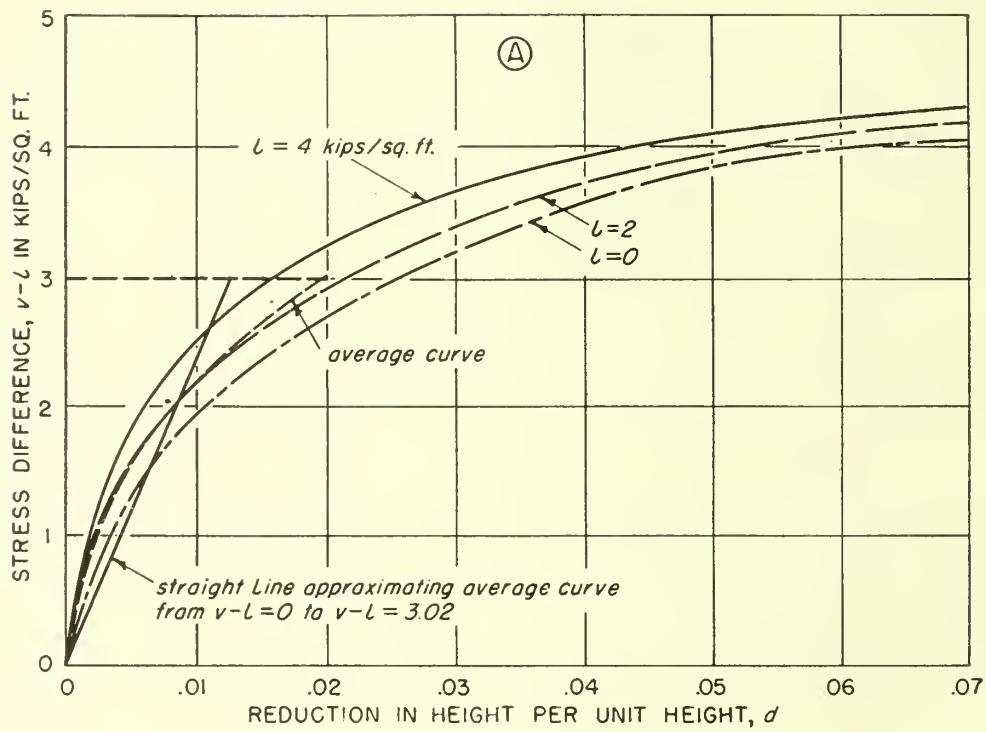
The average normal stress is plotted in figure 6 as point O on the horizontal or normal stress axis. A point representing the normal and shear stress on one plane is plotted such as point A in figure 6 for the plane perpendicular to the radial direction. Point A has an abscissa of 0.026 and an ordinate of 0.040. With point O as a center and distance OA as radius, a semicircle is drawn. Any point B on this semicircle (Mohr's circle of stress) represents the stresses acting on a plane making an angle of one-half AOB with the plane of the stresses represented by point A. The intersections of the semicircle with the normal stress axis determine the principal stresses at the point. These principal stresses act in the two mutually perpendicular directions in which there are no shear stresses. The maximum shear stress is equal to radius OA and acts in a direction at 45 degrees to the direction of the principal stresses.

In figure 6 only two dimensions have been considered, whereas there is always a third principal stress. For the case of an axially symmetrical load such as is considered here, this is the tangential normal stress and has an intermediate value.

TRIAXIAL COMPRESSION TEST

The stresses at a point in a loaded soil mass may be simulated in a cylindrical sample by applying normal stresses to its faces by means of an apparatus such as shown in figure 7 and in the cover illustration. The stress on the ends of the cylinder corresponds to the major principal stress. The stress on the curved face corresponds to the minor principal stress. The intermediate principal stress is the same as the minor principal stress, which, although an arbitrary condition, is satisfactory since it is the severest condition possible and has a minor effect. While compressive stresses are applied, the sample fails by shearing so that the test has been called both triaxial compression and triaxial shear.

To evaluate a subgrade material by means of this test, a sample may be taken from the subgrade in an undisturbed condition or prepared so as to simulate the worst conditions expected to obtain during the life of the struc-



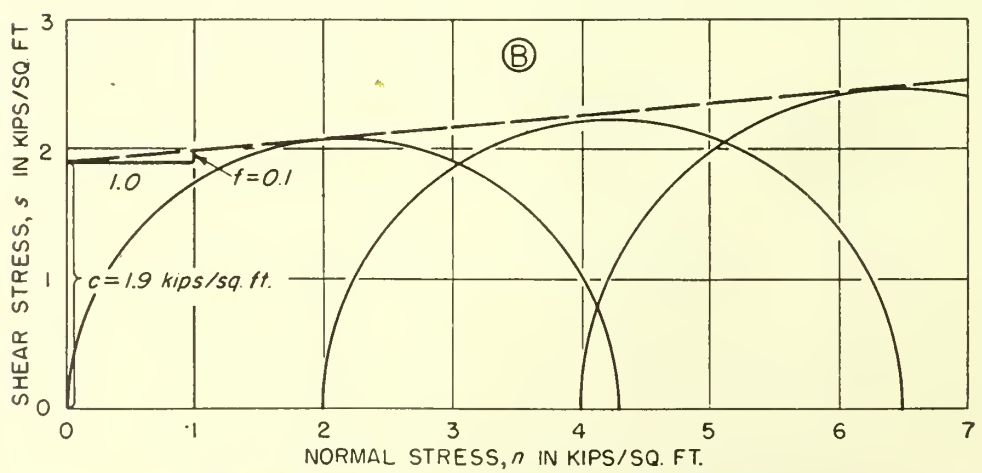


Figure 8.—Plot of triaxial compression test results.

ture and for which it is economical to design. Evaluation of field conditions is a problem in itself, involving observations at various locations of moisture, density, and temperature variations with time; and analysis of their relation to soil and pavement properties, climate, topography, ground water, and loading.

The sample is placed in a rubber membrane which is clamped to two rigid end plates, the lower of which is connected to a drain. If desired, the drain may be closed to prevent drainage from the sample. The lucite cylinder and loading head are assembled and tightened, and the head of the testing machine is brought just in contact with the piston. A constant pressure is applied to the chamber around the sample and is designated lateral pressure, l. The reaction indicator of the testing machine is set to zero. The piston is displaced at a rate of 0.04 inch per minute and the reaction and displacement recorded. The test is continued until the reaction becomes constant or diminishes. The vertical pressure, v, minus the lateral pressure, l, is calculated from the reaction, Q, the reduction in height per unit height, d, and the initial cross-sectional area of the test sample, A, by the relation

$$v-l=\frac{Q-dQ}{A}$$

Separate samples are tested with different lateral pressures to evaluate the effect of this variable. Its effect on the maximum reaction may also be determined from a single sample for materials which are not brittle by retesting the same sample at a higher lateral pressure.

The results of tests on three samples taken from a compacted clay subgrade are shown in table 4. For the three values of l, $v-l$ is plotted against d in figure 8A. The reduction in height due to l for $v-l=0$ represents volume change (0.2 percent for $l=2$ and 0.3 percent for $l=4$) and is not plotted in figure 8A, which is intended to represent the effect of distortion or lateral displacement. The curves for lateral pressures of 0, 2, and 4 kips per square foot are not far apart and an average curve will be used to calculate a modulus of elasticity.

For sand, $v-l$ is approximately proportional to l, making the construction of one effective curve a function of l and therefore a function of the field loading to be considered.

Table 4.—*Report of triaxial compression tests*

REDUCTION IN HEIGHT, d

Vertical minus lateral pressure, $v-l$ (kips per sq. ft.)	Lateral pressure, l, in kips per sq. ft.		
	0	2	4
	Percent	*Percent*	*Percent*
0[1]	0.0	0.2	0.3
1	.3	.4	.45
2	1.0	1.0	.9
3	2.5	2.3	1.8
4	6.0	5.0	4.5

SUPPLEMENTARY TEST DATA

Maximum $v-l$, kips per sq. ft.	4.3	4.5	5.0
Corresponding d, percent	10	16	17
Initial density, lb. per cu. ft.	124	126	126
Initial moisture content, percent of dry soil	21.8	22.3	22.5
Initial density (dry weight), lb. per cu. ft.	102	103	103

[1] Lateral pressure applied.

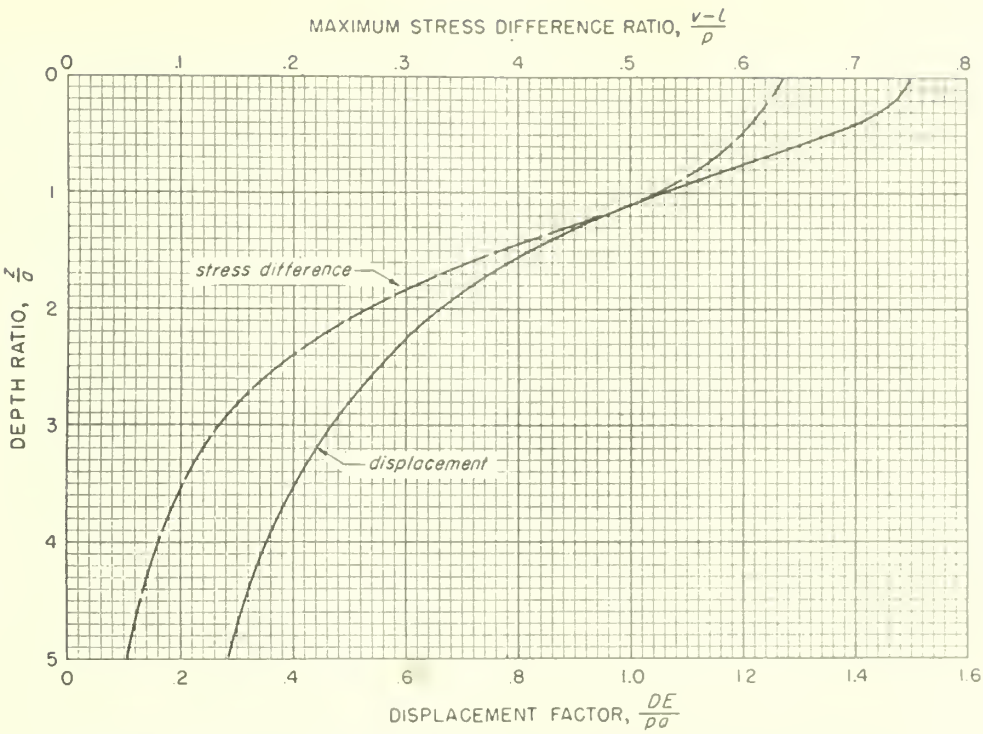


Figure 9.—Vertical displacement and stress difference relation to depth.

Stress semicircles based on the principle of figure 6, using l and the maximum $v-l$ from table 4, are plotted in figure 8B. The minor principal stress is l, and $v-l$ equals the difference in principal stress which is the diameter of the stress circle. The circles have their centers on the horizontal axis at $l+\frac{v-l}{2}$ and pass through l plotted on the same axis. A straight line approximating the envelope of these semicircles defines a relation between the normal stresses and shear stresses on the planes of failure of the test cylinders. (The line is not tangent to all three semicircles due to variations in the samples.) The intercept of this line on the vertical axis is termed the cohesion, c, and its slope is the coefficient of internal friction, f (the angle of internal friction is arc tan f).

THICKNESS OF PAVEMENT AND SETTLEMENT[3]

The vertical displacement factors for points below the center of a uniform pressure applied over a circular area are plotted in figure 9 for Poisson's ratio=0.5. The maximum stress difference at any depth is also shown. These curves may be used to calculate the maximum displacement due to distortion below any depth for a given load on a cohesive subgrade for which triaxial compression test results are available.

For example, assume a load $P=10$ kips, with a unit pressure $p=8.64$ kips per square foot (60 pounds per square inch), and calculate the displacement at a depth of 1 foot for a subgrade represented by the test results shown in table 4 and figure 8.

The equivalent radius, a, of the area over which the load is applied is calculated as follows:

$$a=\sqrt{\frac{P}{\pi p}}=\sqrt{\frac{10}{3.14\times 8.64}}=0.607 \text{ feet.}$$

At a depth of 1 foot, $\frac{z}{a}=\frac{1}{0.607}=1.65$.

From figure 9, for $\frac{z}{a}=1.65$, $\frac{v-l}{p}=0.35$ and $\frac{DE}{pa}=F$ (as defined in table 2) $=0.78$.

Then $v-l=0.35p=0.35\times 8.64=3.02$ kips per square foot.

In figure 8A, a straight line is drawn to approximate the average of the test curves from $v-l=0$ to $v-l=3.02$ by making the area between the vertical axis and the straight line equal to the area between this axis and the average curve. The slope of the straight line is taken as the effective modulus of elasticity, E, for the subgrade. Using the ordinate on this line at $d=0.01$ gives $E=\frac{2.35}{0.01}=235$ kips per square foot. Since $\frac{DE}{pa}=0.78$, the displacement at a depth of 1 foot is $D=\frac{0.78\ pa}{E}=\frac{0.78\times 8.64\times 0.607}{235}=0.0174$ foot or 0.21 inch.

If an allowable displacement is determined from field observations, calculations similar to the foregoing would determine the depth, z, in the subgrade at which this displacement would occur. Since E depends on the depth, z, it is generally most convenient to calculate D for various assumed values of z and plot z

[3] Settlement due to volume reduction could be calculated separately. However, since volume reduction strengthens the soil, the resulting settlement is usually not critical in pavement thickness design for distributed traffic and is not calculated here.

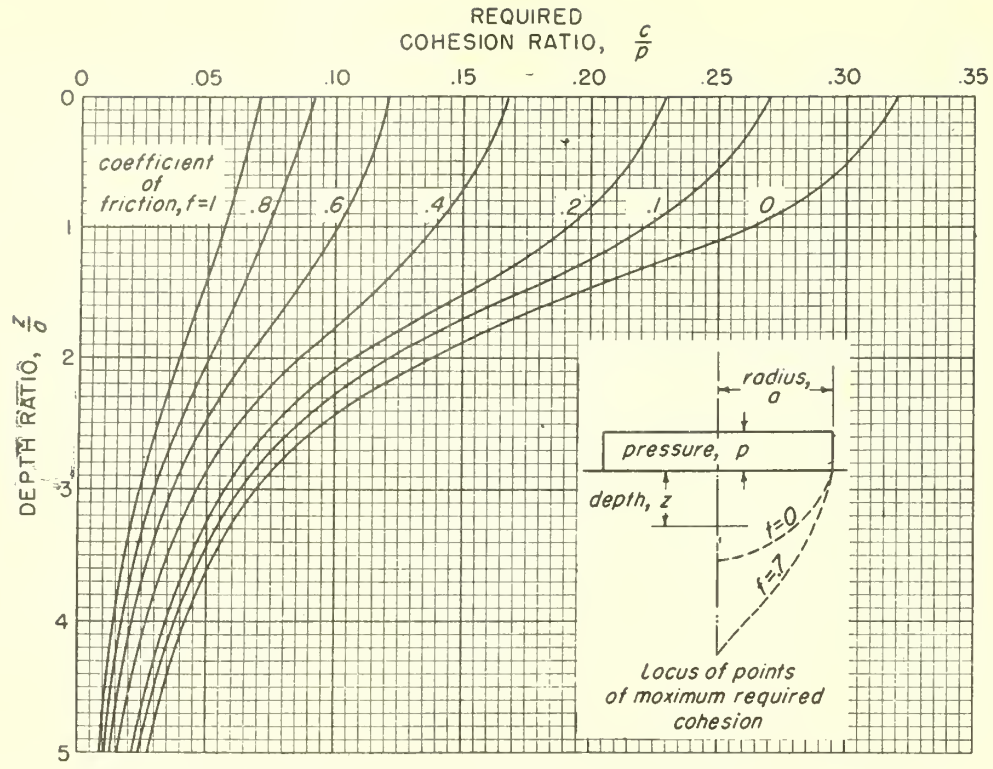


Figure 10.—Depth required to prevent overstress of a point.

against D to determine z for given values of D. Thus, for the above conditions, at $z=1.5$ feet, $D=0.110$ inch, and at $z=2$ feet, $D=0.064$ inch. For an assumed allowable displacement of $D=0.1$ inch, $z=1.6$ feet by interpolation.

For a flexible pavement with a modulus E_p equal to that of the subgrade, this depth would be taken as the required pavement thickness (surface plus base). If E_p is greater than E, the pavement thickness, t, could be calculated from the formula (*13*):

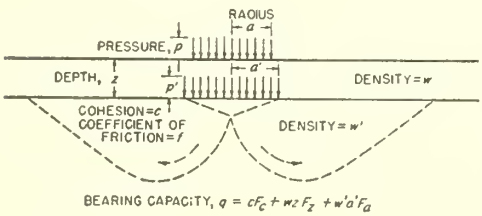
$$t=z\sqrt[3]{\frac{E}{E_p}}$$

For several layers with different moduli, each layer can be considered as an equivalent thickness of subgrade.

For example, the Kansas Highway Department has used the above analysis (*14*) for calculating pavement thickness using $E_p=2{,}160$ kips per square foot and $D=0.1$ inch. Using this value of E_p and $E=330$ corresponding to $z=1.6$ feet, the required pavement thickness is

$$t=1.6\sqrt[3]{\frac{330}{2160}}=0.855 \text{ feet}=10.3 \text{ inches.}$$

For granular materials the modulus varies appreciably with the lateral pressure, making it difficult to select an effective value of the modulus. For example, a base-course material conforming to the A.A.S.H.O. specifications gave a modulus of 500 kips per square foot with a lateral pressure of 1 kip per square foot, and 1,000 kips per square foot with a lateral pressure of 2 kips per square foot. Field correlations may make it possible to determine an effective lateral pressure to use in evaluating such materials.



f	F_c	F_z	F_a
0	74	00	00
0.1	9	08	01
0.2	14	21	04
0.3	19	43	12
0.4	27	8	30
0.5	36	14	66
0.6	53	24	13
0.7	75	40	26
0.8	106	67	51
0.9	156	108	102
1.0	224	172	192

Figure 11.—Bearing capacity under a circular loaded area.

THICKNESS OF PAVEMENT AND OVERSTRESS OF A POINT

By plotting the stress ratios from table 1 in the manner shown in figure 6, the various combinations of cohesion and friction required to prevent overstress at points at various depths were calculated (*15*) and are plotted in figure 10. Thus, for $p=8.64$ kips per square foot and $\frac{z}{a}=1.65$ as in the previous example, and with $f=0.1$ from figure 8*B*, figure 10 shows that $\frac{c}{p}=0.16$. Then the cohesion required to prevent overstress of any point in the subgrade below a depth of 1 foot is $c=0.16\times 8.64=1.38$, which is to be compared with the test value of 1.9 from figure 8*B*, giving a factor of safety of $\frac{1.9}{1.38}=1.38$.

For a factor of safety of 1, $\frac{c}{p}=\frac{1.9}{8.64}=0.22$, which from figure 10 gives $\frac{z}{a}=1.0$ or $z=1.0\times 0.607=0.607$ feet or 7.3 inches. Neglecting pavement rigidity, this is the minimum allowable thickness. Comparison of this design method with field observation has been reported from Great Britain (*16*), indicating its applicability to cohesive subgrades.

Overstress at a point occurs at a considerably lower pressure than the bearing capacity which corresponds to total failure for a single loading; however, a large number of repetitions of a load which causes overstress at a point in a clay or loose sand may eventually cause failure by progressive deterioration. In dense sands this criterion is invalid because such a material tends to expand before failure so that the stresses are redistributed from a region which is approaching overstress until the bearing capacity is reached, whereupon the sand fails suddenly in a manner typical of brittle materials.

THICKNESS OF PAVEMENT AND BEARING CAPACITY

The vertical pressure transmitted through the pavement may be compared with the bearing capacity of the subgrade (*17*) as indicatedi figure 11. The ratio of the maximum vertical pressure, p', at any depth to the pressure applied at the surface, p, is plotted in figure 12 from table 1. The pressure distribution on horizontal planes is not uniform but may be approximated for the present purpose (see fig. 11) by a uniform pressure equal to p' distributed over an area of radius a' such that the total load at any depth is equal to the load applied at the surface. The ratio $\frac{a'}{a}$ is given in figure 12.

For illustration, again take $z=1$ foot, $a=0.607$ foot, $\frac{z}{a}=1.65$, $p=8.64$ kips per square foot, $f=0.1$, and $c=1.9$ kips per square foot; find the density of the subgrade $w'=125$ pounds per cubic foot or 0.125 kips per cubic foot, which is the average initial density given in table 4; and assume the density of the pavement $w=0.140$ kips per cubic foot. From figure 12, for $\frac{z}{a}=1.65$, $\frac{a'}{a}=1.63$ and therefore $a'=1.63\times 0.607=1.0$ foot. Also, from figure 12, $\frac{p'}{p}=0.375$ and therefore $p'=0.375\times 8.64=3.24$ kips per square foot. Now the bearing capacity, q, may be computed from the formu-

Table 5.—*Effect of moisture content of clay subgrade on calculated pavement thickness*

Moisture content	Pavement thickness for 10-kip wheel load, when design criterion is—		
	Displacement	Point overstress	Bearing capacity
	Inches	*Inches*	*Inches*
14	0	0	0
19	4.2	0	4.5
23	9.8	8.6	8.0
26	12.5	15.1	13.6

la and factors given in figure 11. Thus, for $f=0.1$, $F_c=9$, $F_z=0.8$, and $F_a=0.1$, and $q=1.9\times9+0.140\times1\times0.8+0.125\times1.0\times0.1=17.2$ kips per square foot. Now q may be compared with p', giving a factor of safety against total failure of the subgrade of $\frac{17.2}{3.24}=5.3$.

For a given factor of safety, the required depth may be calculated by trial. Thus for a factor of safety of 3, the required thickness of nonrigid pavement or required effective depth of cover is $z=7$ inches.

For a cohesive material with a low value of f, as in the example above, the bearing capacity is approximately cF_c and the depth and size of loaded area have only a small effect. For sand the reverse is true. Thus, for a moderately compact sand we might have $c=0$, $f=0.8$, and $w'=0.105$ kips per cubic foot, so that with the above dimensions $q=0+0.140\times1\times67+0.105\times1.0\times51=14.8$ kips per square foot. Values of q would be higher for greater values of f which would result from increased compaction.

This method of design has been used extensively (*18*) except that the bearing capacity of the subgrade is here calculated from triaxial compression test results instead of being assumed or derived from field loading tests.

EFFECT OF MOISTURE

The effect of moisture content of a single clay subgrade material on pavement thicknesses calculated by the three methods outlined above is shown in table 5. The calculated thickness depends, of course, upon the allowable settlement or required factor of safety against overstress, which in turn can only be determined by correlation of analysis of test results with pavement performance. In table 5, $P=10$ kips and $p=8.64$ kips per square foot. For the displacement method, $D=0.2$ inch and $E_p=2{,}160$ kips per square foot. For point overstress, a factor of safety of 1 is used. For bearing capacity, a factor of safety of 3 is assumed.

These analyses consider vertically applied pressures only. Therefore, the zero pavement thicknesses calculated for a moisture content of 14 percent do not indicate that no cover is required to take care of abrasive stresses. Furthermore, if such a soil were left unloaded in a humid climate, it would become much wetter and weaker. In fact, the pavement may have more effect on the strength of the subgrade than it does on the stresses transmitted to the subgrade. The moisture content of 26 percent was obtained by compacting samples to the A. A. S. H. O. maximum density and optimum moisture content and then allowing them to sorb water while confined by an all-around pressure equivalent to a head of 1 foot of water.

Table 6.—*Effect of type of subgrade on variation of pavement thickness with wheel load based on bearing capacity*

Wheel load (kips)	Pavement thickness on—	
	Clay subgrade	Sand subgrade
	Inches	*Inches*
5	9.5	8.3
10	13.6	10.0
15	16.6	10.9
20	19.2	11.5
30	23.6	12.3
60	33.3	13.3

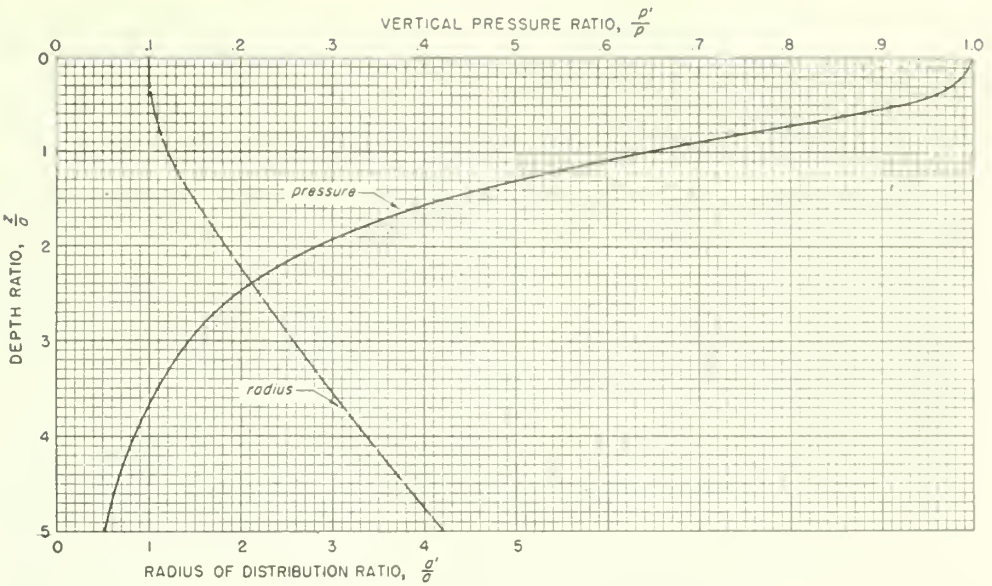


Figure 12.—Vertical pressure distribution.

EFFECT OF LOAD

For a constant unit pressure, the variation of pavement thickness with load calculated by the displacement method depends upon the variation of the allowable settlement with the wheel load. For instance, if the allowable D is assumed proportional to a, then t is proportional to $\sqrt{P}$. For D independent of the load, t increases much more rapidly with increased load.

For a constant factor of safety in the point overstress method, t is proportional to $\sqrt{P}$ if a constant strength of soil is assumed. As the pavement thickness increases, however, it may be expected that the minimum seasonal strength of the subgrade will increase so that t would actually increase less rapidly than $\sqrt{P}$.

Using a factor of safety of 3 against bearing capacity for the clay at 26 percent moisture with $c=1.1$ and $f=0$, and for sand with $c=0$ and $f=0.8$, pavement thicknesses for various wheel loads and $p=8.64$ kips per square foot are given in table 6. It may be seen that for the clay t is proportional to $\sqrt{P}$, but for the sand the rate of increase is much smaller.

BRAKING STRESSES

In addition to vertical pressures, the stopping of vehicles causes shear stresses, s, on the surface in the direction of travel. The magnitude of s depends upon the rate of deceleration and is limited by the product of the vertical pressure and the coefficient of friction between the tire and the surface.

The stresses transmitted from such a surface shear stress may be of use in determining the required thickness and strength of the surface course. For instance, the horizontal shear stress, s_h, near the contact of a surface course with a base course may be a factor in the required thickness of surfacing or the required strength of the materials near the plane of contact. For a uniform shear stress, s, applied over a circular area at the surface as shown in figure 13, the maximum values of s_h are in the vertical plane through the center of the circle and oriented in the direction of travel.

It so happens that the ratio $\frac{s_h}{s}$ is the same numerically as $\frac{p_r}{p}$ for Poisson's ratio $=0.5$ for a vertically applied load (*9*) as tabulated in table 1. Thus, for $\frac{z}{a}=0.5$, the maximum value of $\frac{s_h}{s}$ is 0.374 at $\frac{r}{a}=0$, so that the maximum shear stress in a horizontal direction at the depth $z-0.5a$ has been reduced to 37 percent of its value at the surface.

The normal stress on a horizontal plane, p_s, is compressive in front of the area to which shear stresses are applied but tensile behind the center of the loaded area. The ratio $\frac{p_s}{s}$ is equivalent numerically to $\frac{s_r}{p}$ given in table

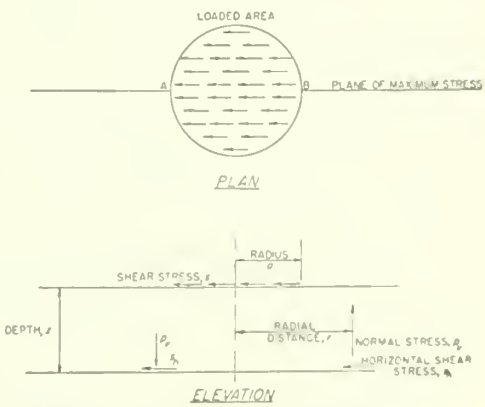


Figure 13.—Stresses from applied shear stress.

1. The maximum tensile stress is at the edge of the loaded area (point B in fig. 13) where $\frac{p_z}{s}=0.318$. At a depth $\frac{z}{a}=0.5$, the maximum value of $\frac{p_z}{s}=0.262$, as shown in table 1 at $\frac{r}{a}=1.0$. The horizontal normal stress developed by an applied shear stress is likewise compressive in front of the loaded area and tensile behind it. Its magnitude is theoretically infinite at points A and B.

The stresses transmitted from a combination of vertical and shear loads may be calculated as the sum of corresponding stresses calculated for each load separately.

The maximum resistance of a surface to horizontal displacement is the sum of its horizontal bearing capacity, Q_h, and the total shearing resistance, S_q, at its lower boundary. For the shear loading considered above, Q_h is approximately $4\ acz+1.4\ cz^2$, where c is half the compressive strength of a cohesive surface; and $S_q=s_q\pi a^2$ where s_q is the unit shearing resistance at the lower boundary. Equating Q_h+S_q to the applied shear force, $s\pi a^2$, and solving for z gives approximately $z=\frac{2}{3}\ a\ \frac{s-s_q}{c}$ as the thickness of surface required to prevent failure by horizontal displacement.

The value of s cannot exceed c because skidding is imminent for $s=c$. For the worst condition, $s_q=0$ and $s=c$, so that $z=\frac{2}{3}\ a$. Then, for a 10-kip load with $a=0.607$ foot, $z=\frac{2}{3}\times 0.607=0.4$ foot or 5 inches. With c greater than s or with an appreciable value of s_q, the required thickness would be reduced. Thus, for the above loading and $c=2s$, $z=2.5$ inches for $s_q=0$ or $z=0.5$ inch for $s_q=0.8s$. If the surface is laminated, the same analysis could be applied to the upper layer of the surface.

NOTATION

a ---- radius of loaded area
a' ---- radius of area such that total load at any depth equals load applied at surface
A ---- initial cross-section area
c ---- cohesion
d ---- reduction in height per unit height
D ---- displacement
E ---- modulus of elasticity of subgrade
E_p ---- modulus of elasticity of pavement
f ---- coefficient of internal friction
F ---- displacement factor
F_a, F_c, F_s ---- bearing capacity factors
l ---- lateral pressure
n ---- normal stress
p ---- pressure on loaded area
p' ---- maximum vertical pressure
p_r ---- radial normal stress
p_t ---- tangential normal stress
p_v ---- vertical normal stress from applied shear stress
p_z ---- vertical normal stress from applied vertical pressure
P ---- total load
P' ---- load per unit length
q ---- bearing capacity per unit area
Q ---- reaction
Q_h ---- horizontal bearing capacity
r ---- horizontal radial distance from load axis to element
R ---- radius of rigid solid cylinder
s ---- shear stress
s_q ---- unit shearing resistance
s_h ---- horizontal shear stress
s_r ---- radial shear stress
s_z ---- vertical shear stress
S_q ---- total shearing resistance
t ---- pavement thickness
v ---- vertical pressure
w ---- density of pavement
w' ---- density of subgrade
z ---- depth to element from loaded surface
μ ---- Poisson's ratio: ratio of lateral to axial strain in a simple compression test

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Scarifying hardened soil-cement base course for reconstruction

Experimental Soil-Cement Base Course in South Carolina

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

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In the 8-year study of soil-cement base courses described in this article it was found that, under the conditions encountered, the durability of the base courses increased proportionally with the percentage of cement used. As expected, performance of soil-cement bases on friable soils was better than that on soils of plastic nature. The conclusion that admixtures of soil and granular material alone performed as well as or better than soil-cement mixtures of equal cost is not surprising, in view of the long history of successful use of stone-stabilized base courses.

FROM 1932 to 1935 the State of South Carolina constructed several experimental sections of highway using portland cement as an admixture with the soils existing in the roadbed to serve as base course for light bituminous surfaces. The excellent results obtained from that early construction contributed largely to the increase in use of soil-cement base courses throughout the State. They served to develop and improve construction procedures and control methods, and furnished data useful for determining cement requirements for different types of soils. As further information became available from the individual sections, it was evident that additional studies were necessary to aid in solving problems relative to base thickness, quantity of cement, and method of curing. Accordingly, the South Carolina State Highway Department in cooperation with the Public Roads Administration constructed an experimental road during the summer of 1938 to study the influence of these variables on the service behavior of the soil-cement base course.

This report describes the construction of the experimental road and presents conclusions drawn from study and observation of its performance under traffic for about 8 years. The 6-mile road crosses rolling country and carries an average traffic of about 500 vehicles per day. Subgrade soils encountered were all of silt-clay material. Twenty-two sections of soil-cement base course were constructed, with thicknesses of 4, 6, and 8 inches, and with cement contents varying from 3 to 11 percent. In addition, two sections were built without cement—in these, stone screenings and crushed stone were added to the soil. Eight of the soil-cement sections were cured with the aid of either tar or emulsified asphalt surface application.

CONCLUSIONS

The conclusions drawn from observation of the experimental road are stated below. They are necessarily confined to the conditions encountered in this investigation.

Portland cement used as an admixture was found to be effective in altering the physical properties of the soil, and this alteration was a major factor in the success of the soil-cement base. One important change effected

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Figure 1.—Soil-cement base-course construction: (A) *Scarifying the roadbed;* (B) *pulverizing with a disk harrow;* (C) *shaping the subgrade;* (D) *spreading cement;* (E) *applying water; and* (F) *wet mixing with a disk harrow.*

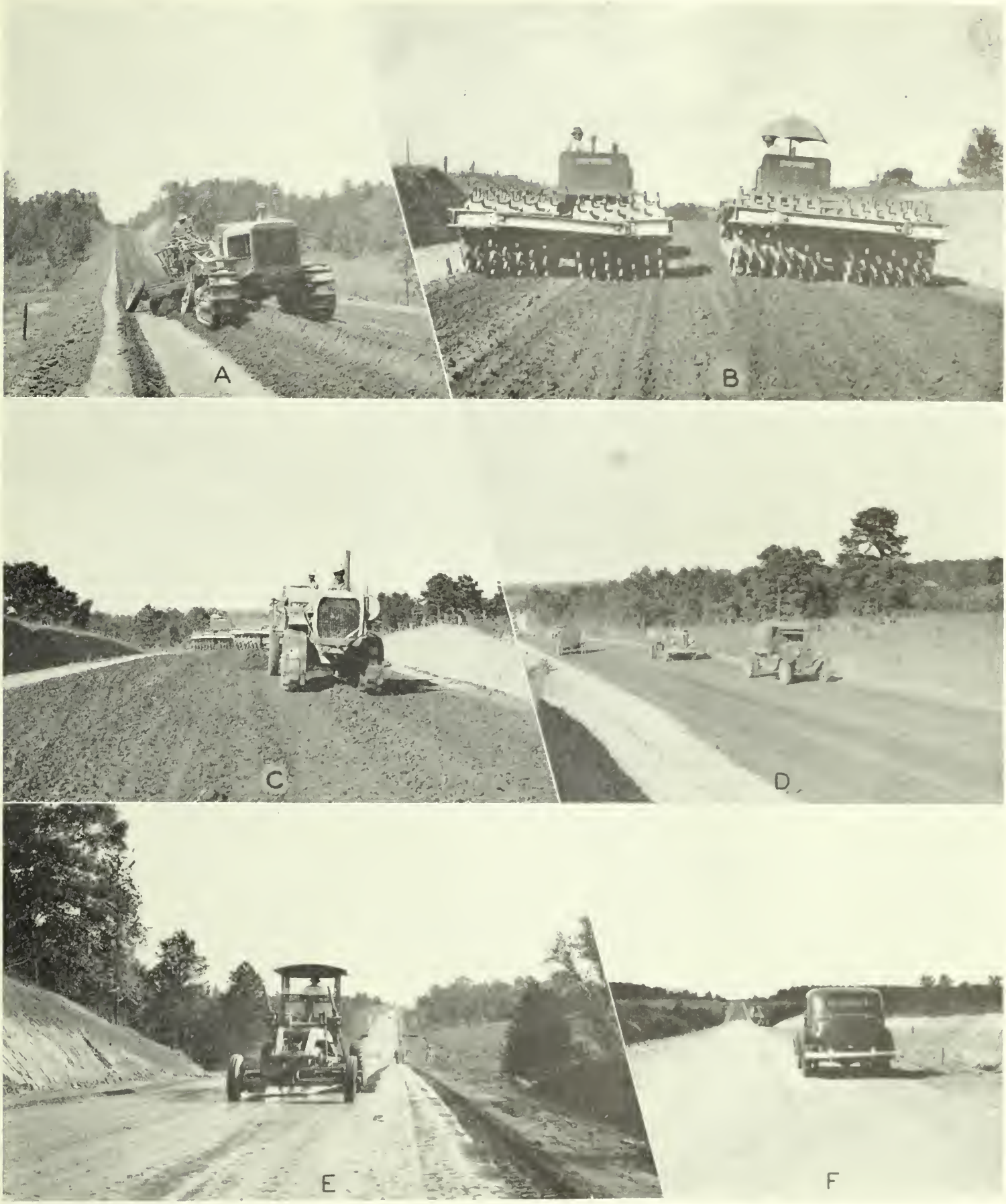


Figure 2.—Soil-cement base-course construction (continued): (A) Wet mixing with a blade grader; (B) compacting; (C) shaping the compacted mixture; (D) rolling with trucks; (E) final blading; and (F) the finished base course.

Table 1.—*Details of experimental sections*

Section No.	Length	Soil classification group	Group index range	Design thickness	Cement content by weight	Curing aid
	Feet			*Inches*	*Percent*	
1 [1]	1,320	A–7	9–13	6	7	None.
2 [1]	1,320	A–7	13–18	6	9	Do.
3	1,320	A–5, A–7	8–13	6	5	Do.
4	1,320	A–7	13–19	6	7	Do.
5	1,320	A–7	14	6	9	Do.
6	1,320	A–7	9–14	6	11	Do.
7	1,320	A–7	11–14	6	3	Tar.
8	1,320	A–6, A–7	4–10	6	5	Do.
9	1,320	A–6	2–8	6	7	Do.
10	1,320	A–7	8–20	6	9	Do.
11	1,320	A–7	14–19	6	[2]	None.
12	1,320	A–4, A–6, A–7	6–15	6	[3]	Do.
13	1,258	A–4, A–6	4	6	3	Emulsified asphalt.
14	1,320	A–4, A–7	2–14	6	5	Do.
15	1,320	A–4	1–7	6	7	Do.
16	1,320	A–4, A–6	4–8	6	9	Do.
17	1,320	A–7	13–20	6	3	None.
18	1,320	A–5, A–6	8–10	6	5	Do.
19	1,320	A–4, A–6	6–7	6	[4] 7	Do.
20	1,320	A–6, A–7	7–11	6	9	Do.
21	1,320	A–4, A–7	4–10	4	5	Do.
22	1,200	A–4	4–5	4	7	Do.
23	1,440	A–4, A–7	7–11	4	9	Do.
24	1,919	A–4, A–6, A–7	3–11	8	3	Do.

[1] Sections 1 and 2 were scarified, pulverized, and recompacted at optimum moisture content about 3 weeks after completion of original base-course construction.
[2] Admixture of stone screenings applied at rate of 300 pounds per square yard.
[3] Admixture of stone screenings and crusher-run stone applied at rate of 300 pounds per square yard.
[4] Rain interfered with original construction. The top 2 inches were pulverized, mixed with 6 percent additional cement, and recompacted.

by the addition of cement was a reduction in the plasticity indexes of the soils.

The addition of cement to the soil increased the durability of the base course, as observed in field performance and as measured in the laboratory by increased resistance to deterioration from wetting and drying and from freezing and thawing. Each increase in the proportion of cement to soil correspondingly increased the quality of performance of the base course.

Soil-cement base courses 6 inches thick, with cement content of 9 percent, and constructed on the more friable soils, gave satisfactory performance for the full 8 years. As might be expected, the soil-cement base courses on friable soils gave better performance than comparable bases on the more plastic soils.

A 3 percent cement content was not sufficient to produce required base-course stability, even when the base thickness was increased to 8 inches.

Soil-cement bases 4 inches thick were inadequate even with cement content as high as 9 percent.

Use of a surface application of asphalt emulsion or tar as an aid in curing the soil-cement base courses proved beneficial, the emulsion producing superior results.

Admixtures of granular material and soil, without cement, gave performance equal to or better than soil-cement mixtures, and at no greater cost. The excellent performance of the stone-stabilized base courses agrees with long experience in which good results have been achieved by this low-cost construction method.

DESCRIPTION OF PROJECT

The experimental road is located on State route 93 in Lancaster County, South Carolina. It begins at a point about 6 miles northeast of Nitrolee and extends in a northeasterly direction for a distance of 6.1 miles to the intersection with route U S 521 at Lancaster.

The road passes through an area of rolling hills containing soils of the Cecil series formed by weathering of the underlying crystaline rocks characteristic of the Piedmont Plateau. Surface soils in this region vary from a light gray or pale yellow fine-sandy loam to a reddish clay loam. Generally, the subsoils consist of red or yellowish-red or mottled yellow-and-red stiff clays. As the underlying rocks are approached, the soil changes to a soft disintegrated rock containing sufficient mica to impart a greasy feel. Quartz veins occur in some places and rock fragments are encountered throughout the entire profile.

A traffic count made in 1941 disclosed that an average of 424 vehicles per day traveled over the southern 17 sections and 800 vehicles per day over the northern end which includes sections 18 to 24. During 6 weeks of Army maneuvers in 1942 between 1,500 and 2,000 vehicles used this road daily, and at one time 500 tanks, varying in weight from 12 to 32 tons, were counted in a 5-hour period. An average of 559 vehicles per day was recorded in a later traffic count made during the week of January 2 to 9, 1942.

Details of the 24 test sections, each approximately one-fourth mile in length, are given in table 1. A soil-cement base was constructed on 22 sections by mixing cement with the natural soil encountered on the grade. Base thicknesses of 4, 6, and 8 inches were employed and the cement content was varied from 3 to 11 percent by weight. No special provision was made for curing the soil-cement base on 14 of the sections. On four sections curing was aided by a light coat of tar applied the day after the base was completed, and the four remaining soil-cement sections received a similar application of asphalt emulsion.

Sections 1 and 2, which contained 7 and 9 percent of cement, respectively, were scarified and repulverized about 3 weeks after they had been completed originally. Following pulverization, the moisture content was brought up to optimum and the base courses were recompacted. This was done to determine the effect of reworking a soil-cement base after hydration was well advanced.

A heavy rain occurred while section 19 was being compacted. The resulting excessive moisture made it impossible to compact the upper 2 inches of the base course. Therefore, on the following morning the upper 2 inches were loosened, pulverized, and mixed with additional cement in an amount equal to 6 percent by weight of dry soil.

Cement was not used in the base courses of sections 11 and 12. Instead, stone screenings and crushed stone were mixed with the soil. In section 11 a mixture of soil with stone screenings, in an amount equal to 300 pounds per square yard, was used. In section 12 the soil was mixed with 150 pounds per square yard of stone screenings following which crushed stone, applied at the rate of 150 pounds per square yard, was incorporated in the upper 2 inches of the soil-screenings mixture. No attempt was made to proportion the stone screenings, crushed stone, and soil in accordance with any particular method of design. The purpose in constructing sections 11 and 12 was to produce a 6-inch base course in which the cost of the stone added would equal the cost of the cement required for an equivalent thickness of soil-cement base course containing 9 percent cement.

The soils [1] occurring in the subgrade were utilized in the construction of the base course. All of them were silt-clay materials of which more than 35 percent passed the No. 200 sieve. They varied from a group A–4 soil with a group index of 1 to a group A–7–5 soil with a group index of 20. The classification and variation in group index of the soils found on each section are indicated in table 1. Analyses of typical soils are given in table 2.

[1] The soil classification used in this report is the one adopted by the Highway Research Board Committee on Classification of Materials for Subgrades and Granular Type Roads and published in the Proceedings of the Highway Research Board, Vol. 25, 1945, pp. 376–384. Reprints of the Classification are available from the Public Roads Administration.

Table 2.—*Results of tests on typical soils*

Sample No.	Sampled from—		Passing sieve—		Liquid limit	Plasticity index	Soil classification group	Group index	Maximum dry density	Optimum moisture
	Section	Station	No. 10	No. 200						
			Percent	*Percent*					*Lb./cu. ft.*	*Percent*
1	15	450	90	41	34	6	A–4	1	112	17
2	20	508	94	66	36	12	A–6	7	113	15
3	17	483	96	79	47	20	A–7–6	13	104	19
4	10	394	100	87	64	33	A–7–5	20	100	23

After a preliminary soil survey had been made, the limits for the experimental sections were established and the percentages of cement to be used were selected arbitrarily. This selection was based primarily on previous experience with soil-cement base construction in South Carolina. Subsequently, three or more samples of soil were obtained from each section for test purposes.

In addition to the mechanical analyses and physical test constants, which were determined in the Public Roads laboratories, compaction and durability tests were performed on these samples in the South Carolina State Highway Department laboratory. In general, durability tests were made on each soil with three different cement contents: that selected for the section, 2 percent above the selected percentage, and 2 percent below. The procedures followed were similar to those later adopted by the American Society for Testing Materials in Methods D 559–44 and D 560–44.

The results of compaction tests on typical soil samples are shown in table 2. Average values of the percentage of loss in weight of soil-cement mixtures after approximately 50 cycles of wetting and drying and of freezing and thawing are shown in table 3.

CONSTRUCTION METHODS

Base construction was started in May and completed in August 1938. A bituminous surface treatment was applied in September 1938 and a re-treatment in August 1939. State forces built the base and surfacing.

Construction of the soil-cement base consisted of scarifying and pulverizing the soils in the roadbed to the required depth and width, spreading the cement, mixing the cement and dry soil, applying and mixing water, compacting the mixtures, and shaping to the desired cross section. The various operations are illustrated in figures 1 and 2.

Shaping of the base commenced when the soil was compacted to the extent that the sheepsfoot rollers penetrated only 1½ inches below the surface. Shaping and compacting were continued until the surface was brought to the desired cross section and the feet of the rollers rode on the surface without penetration. The sheepsfoot rollers were then taken off the road and compaction was completed by rolling with pneumatic-tired trucks. The appearance of the surface after rolling is shown in figure 3*A*.

Surface irregularities and roller marks were removed by blading as soon as the rolling was completed. Approximately one-half inch of material was removed in this manner. This work was done with the motor grader shown in figure 2*E* and also with a light tractor-drawn blade grader. The appearance of the surface after the final blading is illustrated in figures 3*B* and *C*: The surface texture in figure 3*B* is typical of locations where the soil was of a friable nature, while the texture in figure 3*C* is representative of the more plastic red clay soils. The scars in the surface were caused by dislodgement and removal of rock fragments during the final blading.

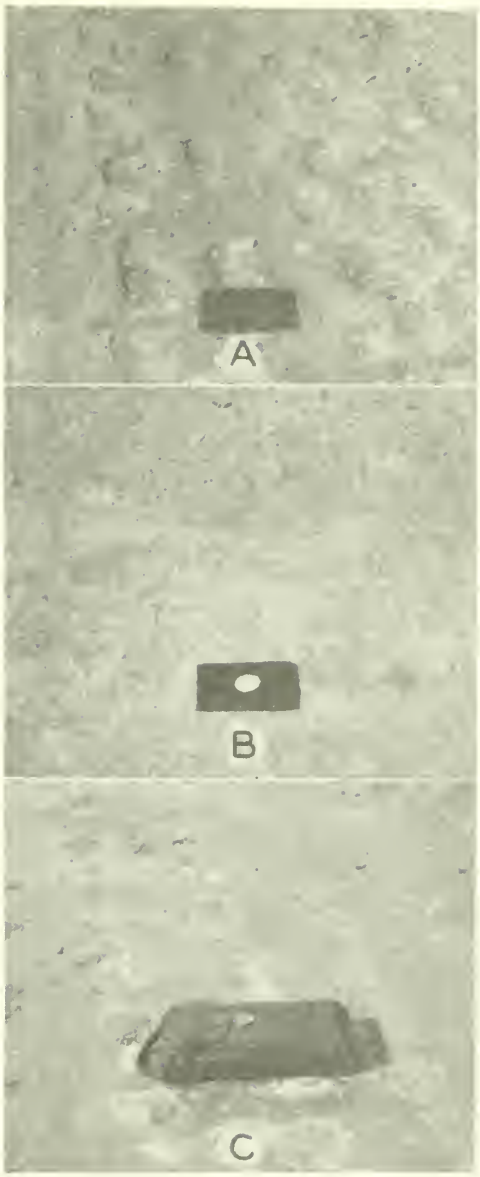


Figure 3.—Appearance of the base after rolling (A) *and after final blading on friable soil* (B) *and on heavy clay soil* (C).

Table 3.—*Results of durability tests of soil-cement mixtures*

Subgrade group and cement content by weight (in percent)	Average loss after approximately 50 cycles of—	
	Wetting—drying	Freezing—thawing
A-4 (group index 1-7) with cement content of:—	*Percent*	*Percent*
3	32	37
5	20	26
7	7	16
9	2	8
11	2	0
A-6 (group index 2-10) with cement content of:—		
3	41	65
5	27	26
7	16	15
9	12	18
11	1	1
A-7-6 (group index 7-18) with cement content of:—		
3	61	78
5	40	39
7	12	22
9	9	11
11	4	11
A-7-5 (group index 8-20) with cement content of:—		
3	100	100
5	46	41
7	20	30
9	5	12
11	1	10

A transverse joint was constructed at the end of each section when the final blading started. The compacted material was first cut away to leave a smooth vertical face for the full depth of the compacted mixture. A wooden header, trimmed to conform to the final cross section of the base, was then staked against the cut face and was not removed until after the adjoining section had been constructed.

In the construction of the joint, difficulty was encountered in getting a uniformly mixed and thoroughly compacted base at the junction without damaging the completed section when the construction equipment turned around. A protective cover of earth from 2 to 3 inches thick was placed over the last 50 feet of the completed section in an attempt to overcome this difficulty. This permitted the equipment to turn on the completed section without producing any damage. However, this procedure did not eliminate the necessity for hand-mixing and tamping at the joint.

The method finally adopted at the turnarounds consisted of blading the materials forward from the joint and making all turns ahead of the completed section, as shown in figure 4. Only pneumatic-tired equipment such as water distributors and the motor patrol were permitted on the completed base course. After each mixing operation was completed, the mixture was spread out uniformly against the joint. Machine-mixing and compacting near the joint were supplemented by hand-mixing and tamping.

On sections 7, 8, 9, and 10 a surface application of tar (RT–2) was used to aid curing, while sections 13, 14, 15, and 16 were similarly treated with emulsified asphalt (South Carolina designation RCO–1). Both materials were used at the rate of about 0.2 gallon per square yard. The tar was applied by a pressure distributor on a surface previously cleaned of loose material. Emulsified asphalt was applied in the same manner as the tar, but no cleaning was necessary as the surface was maintained in a moist condition by sprinkling until the application of the emulsion. The several applications of water amounted to approximately 2.2 gallons per square yard of surface.

The remaining 16 sections were allowed to cure without a protective covering of any kind until the surface course was constructed.

Sections 1 and 2 were torn up and reconstructed about 3 weeks after they were originally constructed. The scarifier tore up the hardened soil-cement mixture in large slabs from 4 to 6 inches thick, as shown in the illustration at the top of page 9. A sheepsfoot roller reduced the slabs to small

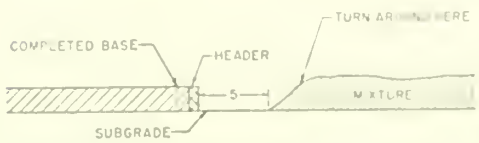


Figure 4.—Construction method at joints.

Figure 5.—(A) Placing stone screenings on section 11; (B) spreading crushed stone on section 12.

clods which were then easily pulverized by disk harrows and blade graders. As no cement was added to the pulverized soil-cement mixture, the dry-mixing operation was eliminated. Otherwise, the reconstruction was performed in the same manner as the original construction.

On section 11 the soil in the roadbed was scarified to a depth of 4 inches for a width of 21 feet. The scarified soil was then pulverized and leveled, and stone screenings were placed in piles on top of the pulverized soil as shown in figure 5*A*. The piles of screenings were spaced so that when spread uniformly over the surface an application of 300 pounds per square yard resulted. The screenings were first spread, then the soil and screenings were mixed thoroughly and uniformly, and finally the mixture was compacted and shaped to conform to the desired cross section.

Table 4.—*Gradations of screenings and crushed stone used in sections 11 and 12*

Material passing sieve size—	Screenings	Crushed stone
	Percent	*Percent*
1½ inch		100
1 inch		96
¾ inch		67
⅜ inch		28
No. 4	100	20
No. 10	84	18
No. 40	50	9
No. 200	18	2

A heavy rain occurred on the night previous to the mixing of the soil with the screenings. For this reason, no water was required for mixing but a great deal of extra disking and blading was necessary in order to dry out the mixture sufficiently to permit compaction. Otherwise, all the operations of scarifying, pulverizing, mixing, compacting, and shaping were performed in the same manner as described for the soil-cement mixtures.

On section 12, a 4-inch layer of the pulverized soil was mixed with stone screenings applied at the rate of 150 pounds per square yard. This mixture was shaped and compacted until the sheepsfoot rollers would penetrate only 2 inches below the surface. Crushed stone was then spread, at the rate of 150 pounds per square yard, from trucks equipped with spreader boxes as shown in figure 5*B*. The crushed stone and the 2-inch uncompacted surface layer of soil and screenings were thoroughly and uniformly mixed, shaped to cross section, and compacted by rolling with a sheepsfoot roller and pneumatic-tired trucks.

Typical gradations of the screenings and crushed stone used in sections 11 and 12 are given in table 4.

Observations of variations in construction procedures, weather conditions, and time required to perform the different operations on each section were made during construction of the base. Important variables have already been described. No correlation was found between weather conditions, or time of mixing, compacting, and other operations, and the condition of the base course. Therefore, since they would serve no useful purpose, weather records and time study data are not presented in this report.

BITUMINOUS SURFACE CONSTRUCTED

A bituminous wearing surface was constructed in September 1938. This consisted of a prime coat of tar at the rate of 0.22 gallon per square yard followed by an application of 0.40 gallon per square yard of hot asphalt (150–200 penetration) covered with crushed stone at the rate of 36 pounds per square yard.

In August of 1939 the experimental road was resurfaced. A tack coat of 0.10 gallon per square yard of cut-back asphalt (RC–1) was applied to the old surface and covered with 42 pounds per square yard of crushed stone. This was followed by two equal applications of sand totaling 40 pounds per square yard. When the last application of sand was dry, cut-back asphalt (RC–1) was distributed over the sand and stone at the rate of 0.90 gallon per square yard. The asphalt and aggregates were mixed by a multiple blade drag, spread, and then compacted with a 5-ton tandem roller.

CONDITION OF BASE DISCLOSED BY CORE BORINGS

At periods ranging from 7 to 40 days following construction of the various test sections, cores were drilled from the base course at 250-foot intervals. Cores were

Table 5.—*Condition of base courses after curing, as revealed by core borings*

Section Number	Designed cement content	Hardening period	Test borings	
			Number	Percent unhardened [1]
	Percent	*Days*		
1	7	30	15	0
2	9	30	18	0
3	5	19	15	60
4	7	18	15	60
5	9	16	18	50
6	11	16	15	73
7	3	40	15	7
8	5	36	15	0
9	7	35	15	0
10	9	34	18	0
11	[2]			
12	[3]			
13 [4]	3	22	15	7
14	5	21	15	0
15 [5]	7	22	18	72
16 [5]	9	19	15	40
17	3	20	12	58
18	5	17	18	6
19	7	17	15	0
20	9	16	15	7
21	5	14	18	28
22	7	24	12	0
23	9	24	18	11
24	3	7	24	13

[1] Percentage of total number of borings where base was not hard for the full processed depth.
[2] Mixture of soil and stone screenings.
[3] Mixture of soil, stone screenings, and crushed stone.
[4] Nonuniform mixing near bridge abutment.
[5] Mixed and compacted after dark.

taken on the center line and about 6 to 8 feet on either side of the center line at each location.

Many of the cores were shattered when the drill struck rock fragments in the soil. In other instances, satisfactory cores could not be obtained because the lower portions of the base-course mixtures had not hardened. For these reasons the thickness of the base courses was measured in the drill hole. The weights and volumes of the sound cores were determined later, in the laboratory, and their densities calculated.

Table 5 shows the total number of borings in each section and the percentage of the locations containing unhardened (dry, friable, and crumbly) material within the processed layer. The table also shows the age of the base at the time of drilling the cores. On all of the sections, except Nos. 3 to 7, wet-mixing during construction was accomplished by disk harrow and blade grader. On sections 3 to 7 only the disk harrow was used.

The greater uniformity of the base in the sections mixed with both disk harrow and blade grader is clearly shown in table 5. Nonuniformly hardened material was encountered at 50 percent or more of the locations drilled in sections 3, 4, 5, and 6, which had been mixed with disk harrow alone. Borings were made when these sections were 16 to 19 days old. Section 7, containing 3 percent cement, was the only other section on which mixing was accomplished solely with the disk harrow. Cores were drilled when this section was 40 days old. At that time nonhardened material was found at only 7 percent of the locations.

The disk harrow and blade grader had been used in combination in the mixing operations on the remaining 17 sections containing cement. On eight of these sections the base was uniformly hardened at all locations where borings were made. These sections were drilled at ages ranging from 17 to 36 days. Nonuniformly hardened material was encountered at some locations in all of the remaining nine sections but in only three of these, sections 15, 16, and 17, was the percentage of occurrence greater than 28 percent. Sections 15 and 16, on which nonuniform hardening was found to be 72 and 40 percent, respectively, of the core drill locations, were mixed and compacted at night with no other light than that furnished by the headlights on the construction equipment. Nonuniformly hardened base was encountered at 58 percent of the locations on section 17, containing 3 percent cement, when cores were drilled 20 days after construction.

The large number of locations where nonuniformly hardened material was encountered in the sections mixed with the disk harrow alone was due apparently to the fact that the disks did not cut deeply enough to distribute the water properly in the lower part of the mixture. This condition was corrected by using the blade grader which produced a more uniform distribution of the water in the mix.

Results of thickness and density measurements are summarized in table 6. A considerable variation from the specified thickness was found in individual cases, ranging from a minimum of 4 to a maximum of 9 inches for the 6-inch sections. However, the

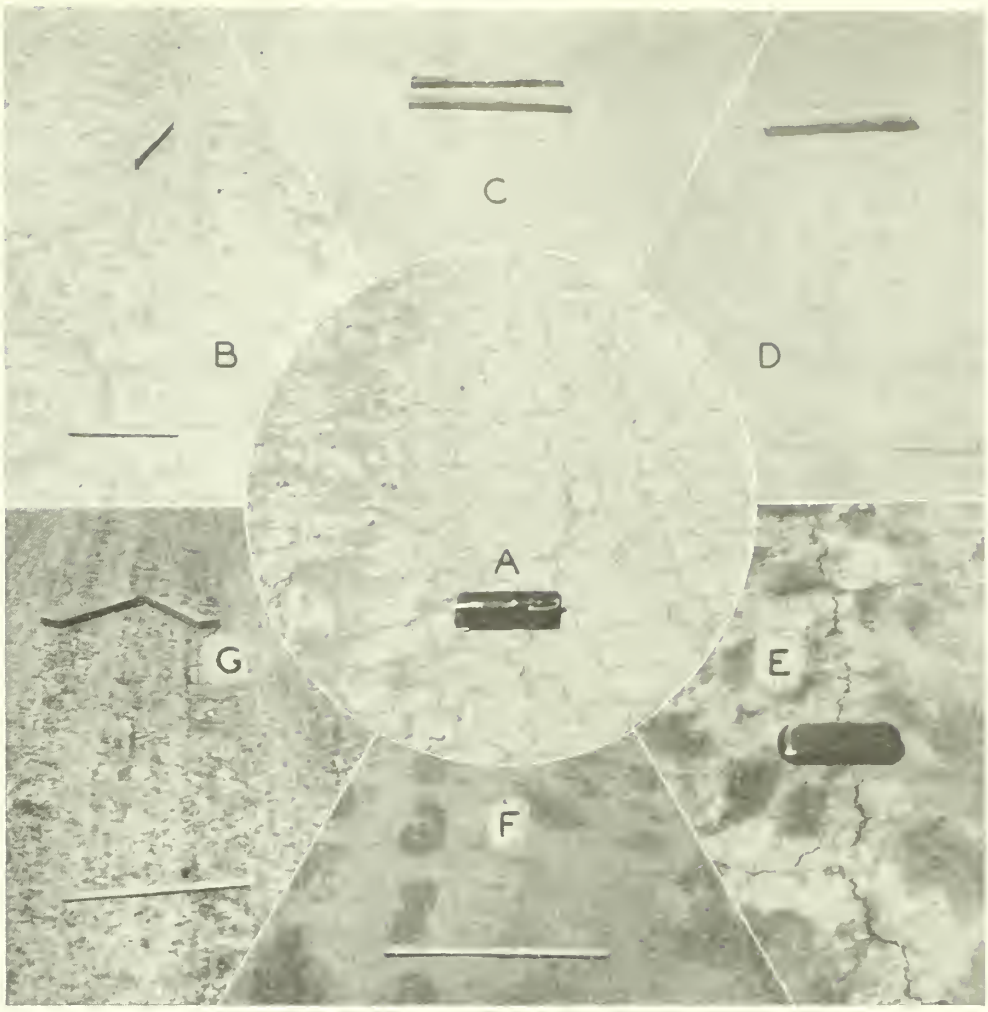


Figure 6.—(A) Shrinkage cracks in the roadbed soil prior to construction. (B) Appearance of the completed soil-cement base course on section 17 (3 percent cement); (C) on section 22 (7 percent cement); (D) on section 19 (7 percent cement) after the top was enriched and reconstructed; (E) on section 1 (7 percent cement) before reconstruction; (F) on the same section after reconstruction; and (G) on a section cured with emulsified asphalt.

Table 6.—*Summary of thickness and density measurements*

Section No.	Designed cement content	Thickness in inches				Density in pounds per cubic foot			Average compaction [1]
		Designed	Measured						
			Avg.	Max.	Min.	Avg.	Max.	Min.	
	Percent								*Percent*
1	7	6	7.0	8.0	5.0	97	103	91	92
2	9	6	7.1	9.0	5.5	97	106	92	97
3	5	6	6.7	8.5	5.2	100	108	92	100
4	7	6	7.0	8.5	5.5	100	107	95	96
5	9	6	6.9	9.0	5.2	102	107	94	101
6	11	6	7.3	9.0	4.3	98	102	91	93
7	3	6	6.5	8.0	5.0	105	109	97	101
8	5	6	6.2	7.0	4.0	100	105	90	96
9	7	6	7.0	8.5	6.0	100	107	96	92
10	9	6	6.4	9.0	5.0	92	97	90	94
11	[2]	6	6.0	7.2	5.2	119	123	117	111
12	[3]	6	6.2	7.0	5.5	122	128	116	103
13	3	6	6.0	9.0	4.0	109	116	105	97
14	5	6	6.0	8.0	4.5	108	111	105	100
15	7	6	6.4	8.5	4.5	98	100	95	94
16	9	6	6.6	8.0	5.5	101	104	96	94
17	3	6	7.3	8.0	6.5	100	100	99	100
18	5	6	6.5	8.0	5.0	97	98	95	93
19	7	6	7.2	9.0	6.0	103	106	99	94
20	9	6	6.1	8.0	5.0	101	104	100	96
21	5	4	5.2	6.5	4.5	100	104	94	93
22	7	4	5.2	6.0	4.5	101	102	100	94
23	9	4	4.5	6.5	4.0	92	97	85	89
24	3	8	7.9	9.5	6.0	101	105	93	93

[1] Average compaction expressed as percentage of average maximum density determined in accordance with Method T-99-38 of the A.A.S.H.O.

[2] Mixture of soil and stone screenings.

[3] Mixture of soil, stone screenings, and crushed stone.

average thickness of any one of this group of 18 sections was never less than 6 inches and the maximum value of average thickness was 7.3 inches. Similar variations from specified thickness were encountered on the 4-inch and 8-inch sections.

Although base-course densities (table 6) varied over a wide range, the average compaction per section, with the exception of section 23, was not less than 92 percent of the average maximum density of the soil-cement mixtures as determined in the laboratory in accordance with Method T–99–38 of the American Association of State Highway Officials. The average compaction for section 23 was 89 percent.

A check on base thickness was made in 1944 by drilling additional cores from each section at locations close to those where the original cores were taken. The individual values varied appreciably from the figures given in table 6. However, the average values for each section showed no significant change from those measured in 1938.

SHRINKAGE CRACKING

Shrinkage cracks that developed in the various sections after the base had been compacted were recorded by means of photographs.

Figure 6*A* shows the roadbed soil before it was scarified and mixed with cement. Typical examples of the cracking that developed in the soil-cement base in many of the sections where no curing aid was provided are shown in figures 6*B* and *E*. Figure 6*C* shows the appearance of section 22, where practically no shrinkage cracking occurred in the soil-cement base.

In general it was observed that as the amount of cement was increased, the amount of shrinkage cracking decreased. Differences in the amount of cracking occurring on sections having the same percentage of cement were probably due to variations in the soil.

The condition of section 19 (7 percent cement), on which the upper 2 inches had been reconstructed with an additional 6 percent of cement, is shown in figure 6*D*. It will be seen that the enriched upper layer was almost entirely free from shrinkage cracks.

The appearance of section 1 after reconstruction is illustrated in figure 6*F*. Comparing figures 6*E* and *F*, it will be seen that as a result of reconstruction the amount of cracking and the crack width had been considerably reduced.

The cracking that occurred in the soil-cement bases on sections 7 to 10, inclusive, where tar was used as a curing aid, was similar in amount and intensity to the cracking observed in the bases of corresponding cement content on the sections that had been allowed to cure without a protective cover.

Shrinkage cracking did not occur in the base courses of sections 13 to 16, where emulsified asphalt was used as a curing aid. Figure 6*G* is representative of the condition of these four sections prior to surface treatment. Unlike the tar, which penetrated to a depth of one-eighth to one-fourth inch and was dry by the following morning, the asphalt emulsion formed a film over the surface and did not dry for about 3 days. Before it dried, the emulsion film could be lifted and peeled off without disturbing the base underneath. After drying, it was securely bound to the base.

Two other factors that probably influenced the behavior of the emulsion-treated sections were the maintenance of the surface in a moist condition by sprinkling until the emulsified asphalt was applied, and the friable character of the soils which, on an average, had lower liquid limits and plasticity indexes than any other four consecutive sections on the experimental road. Sample No. 1, table 2, is typical of the soils in these sections.

No cracking developed in the crushed stone and screenings sections, Nos. 11 and 12, except in one 140-foot length where water from heavy rains had saturated the loose mixture of soil and screenings. The mixture in this portion of the road was bladed back and forth for 5 hours until it was dry enough to roll. The moisture content, even after this period of aeration, was judged to be above optimum.

Table 7.—*Effect of cement on average liquid limits, plastic limits, and plasticity indexes*

Subgrade group and cement content (in percent)	Average liquid limit	Average plastic limit	Average plasticity index
Group A–4 with cement content of:			
0	33	26	7
3	36	32	4
5	35	33	2
7	36	[1] NP	NP
9	34	NP	NP
Group A–6 with cement content of:			
0	36	22	14
3	38	31	7
5	36	32	4
7	40	36	4
9	37	36	1
Group A–7–6 with cement content of:			
0	47	27	20
3	36	29	7
5	38	31	7
7	39	37	2
9	38	36	2
11	38	NP	NP
Group A–7–5 with cement content of:			
0	53	33	20
3	40	30	10
5	38	30	8
7	40	35	5
9	40	37	3
11	37	NP	NP

[1] NP=nonplastic.

EFFECT OF CEMENT ON PLASTICITY

Samples of pulverized soil were taken from each section before cement was added. These samples, representative both of the soil in the soil-cement mixture and of the subgrade soil, were tested to determine their gradations and physical properties. The cores of hardened soil-cement previously described were obtained from the completed base course at approximately the same locations as the pulverized soil samples. After their densities had been determined, the cores were pulverized and the resulting material tested for liquid limit and plasticity index.

In all but 2 of the 66 locations sampled, the plasticity indexes of the soil-cement mixtures were appreciably lower than that of the raw soil. The plasticity indexes of the mixtures and raw soils were equal in the two exceptions noted.

The effects of the various percentages of cement on the average liquid limits, plastic limits, and plasticity indexes of soils of the A–4, A–6, A–7–6, and A–7–5 subgrade groups encountered are shown in table 7.

The cement admixtures were effective in reducing the liquid limits of the raw soils having liquid limits above about 40, while generally producing an increase in those soils having liquid limits below 40. This relation was observed regardless of the percentage of cement.

In illustration, the average liquid limits of the group A–4 and group A–6 soils without cement were 33 and 36, respectively. The same soils with cement admixtures had somewhat higher average liquid limits. The addition of cement to the group A–7–6 and A–7–5 soils, which had average liquid limits of 47 and 53, respectively, resulted in average liquid limits between 36 and 40.

The average plastic limits of all four subgrade groups were increased by the addition of cement except in the case of the group A–7–5 soils with 3 and 5 percent cement, where a slight decrease resulted. In all cases, the average plasticity indexes of the soil-cement mixtures were appreciably less than the corresponding average values for untreated soil. In general, the greater the percentage of cement, the greater was the reduction in the plasticity index, although the addition of 3 percent of cement produced a greater change than any succeeding increment.

OBSERVATIONS FROM CONDITION SURVEYS

Data relative to the service behavior of the various sections following construction of the bituminous surface were obtained in six surveys made in March 1939, March 1940, November 1940, February 1942, May 1944, and April 1946. The condition at the time of the last survey is shown graphically in figure 7. Sections having the same cement content are grouped together in this figure in order to facilitate the study of the several variable factors.

The survey of March 1939, when the road was about 7 months old, disclosed no base-course defects except in section 24 (3 percent cement) where a 75-foot length of the base was destroyed as a result of soft, unstable subgrade caused by seepage. Surface failures consisted almost entirely of pecling and pitting of the thin bituminous mat and some raveling along the edges of the surfacing. Unsatisfactory drainage caused by high shoulders that became soft and deeply rutted contributed largely towards these conditions. Surface irregularities and defects were corrected by the bituminous re-treatment placed in August 1939.

The shoulders were rutted badly at the time of the March 1940 inspection. The surfacing was in good condition throughout except that failures were beginning to occur in the sections containing 3 percent cement.

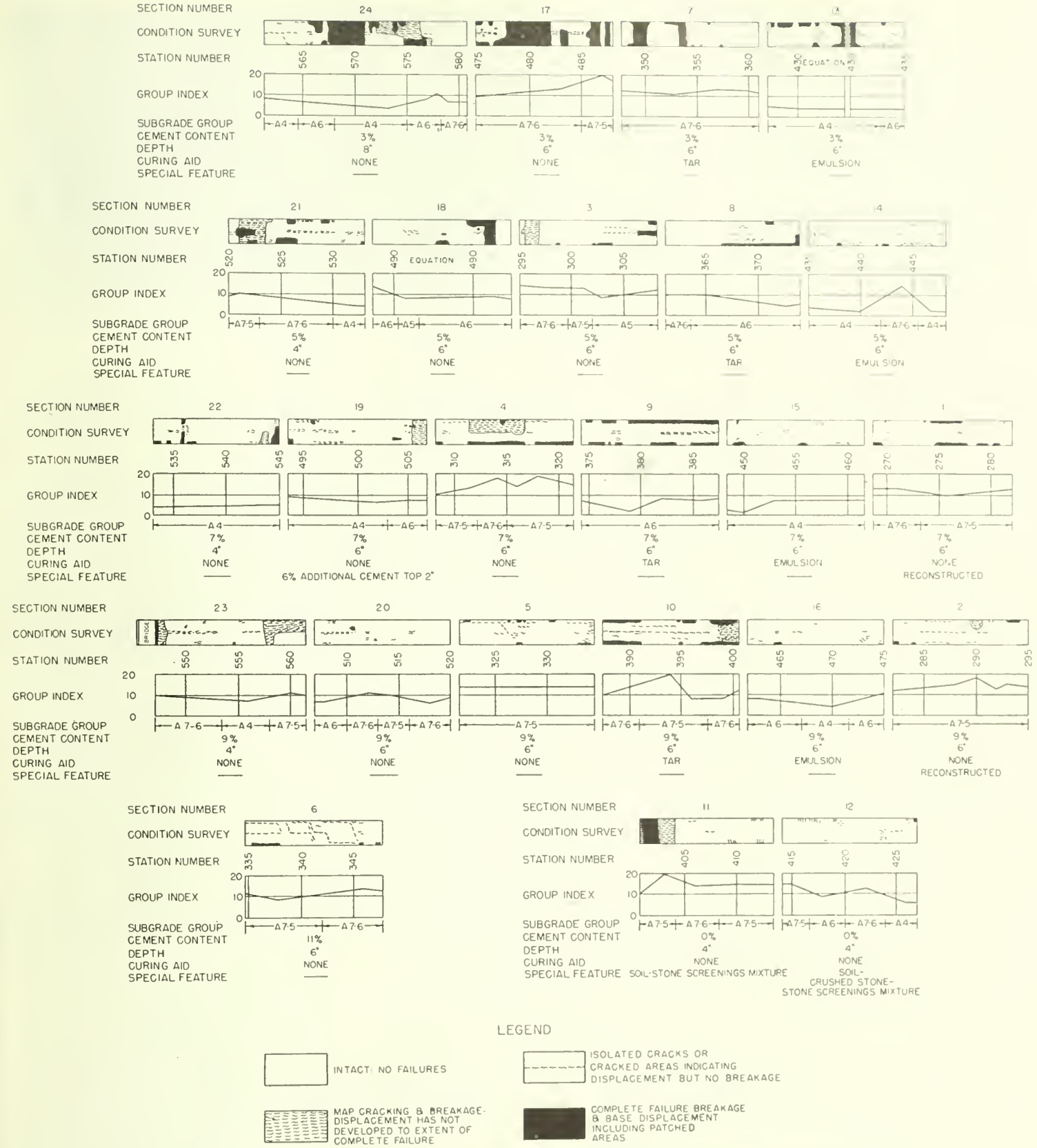


Figure 7.—April 1946 condition survey; sections grouped according to cement content.

The survey made in November 1940 revealed no significant change in the condition of the pavement.

The February 1942 survey showed that after 3½ years of service important failures had developed in the sections containing 3 percent cement. These failures consisted of breakage and displacement. Map cracking was developing in the other sections. There was also evidence that fill settlement was responsible for some of the cracking that occurred. This survey was made approximately 3 months after the completion of extensive Army maneuvers.

By May 1944, failures had increased in number and area to the extent that there was a definite indication of the difference in performance of the various sections. Additional

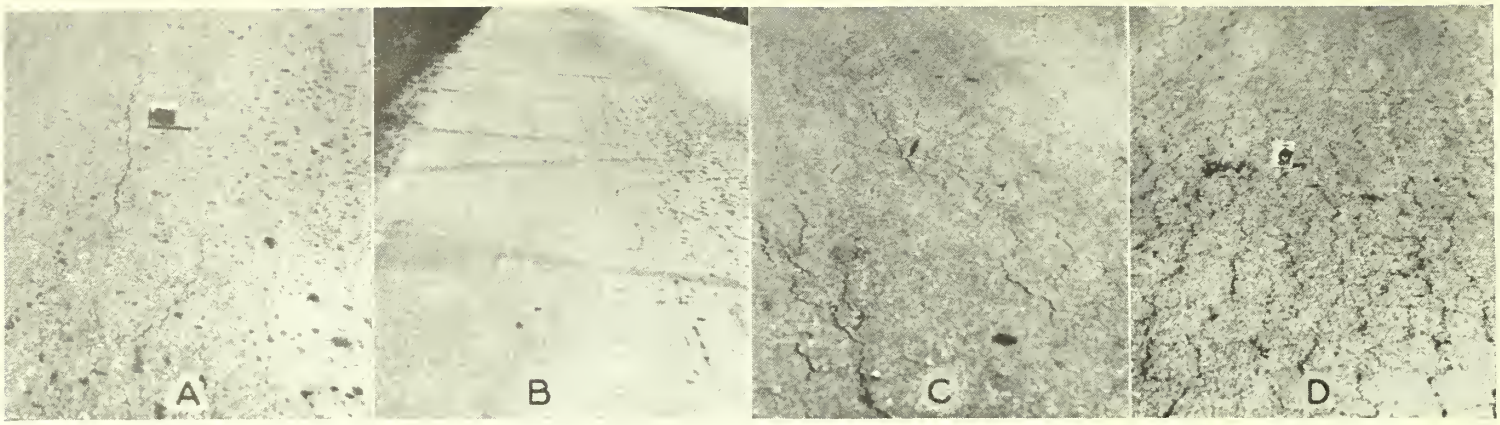


Figure 8.—Pavement performance: (A) *Isolated longitudinal cracks;* (B) *isolated transverse cracks;* (C) *map cracking and breakage; and* (D) *complete failure.*

failures occurred between the surveys of May 1944 and April 1946 but they were less in number and degree than had occurred during the previous 2 years. Most of the maintenance required between 1944 and 1946 consisted of repairing previously patched areas that continued to break up.

The varying conditions of the pavement, after approximately 8 years of service (fig. 7), were classified into four groups as follows:

Intact.—Areas containing no cracks or failures of any kind were classed as intact.

Isolated cracking.—Isolated cracks or cracked areas, indicating displacement but no breakage, are illustrated in figure 8*A* and *B*. This type of cracking was probably the result of very slight displacement produced by traffic or by volume changes in the base and subgrade. Transverse cracks, as shown in figure 8*B*, were evidently caused by shrinkage in the base or subgrade, or both. This condition did not affect the riding qualities of the road. It was important principally because of the fact that additional cracking of the same kind developed into areas of map cracking.

Map cracking and breakage.—In the classification of map cracking and breakage, a condition bordering on complete failure developed (fig. 8*C*) but was not classed as complete failure because displacement was not in evidence and riding qualities were not impaired.

Complete failure.—Complete failure due to breakage and base displacement is illustrated in figure 8*D*. This condition included all areas where the surfacing had been replaced by patches of premixed bituminous materials. The structural weakness of the base at these locations was demonstrated by the continued breaking up of the patches. Breakage was accompanied by displacement of base and surfacing and produced a rough riding surface.

Figure 7 shows general relations between the condition of the road surface and character of soil, base thickness, cement content of base, method of curing, and other factors. Many of the discrepancies in the general trends are attributable to fill settlement. While no measurements of subsidence were made, the effects of fill settlement on bridge approaches were observed soon after the road surface was constructed. Later surveys indicated that gradual settlement was taking place in all the fills, especially those over 10 feet in height, and was definitely a contributing factor in the cracking that developed.

Four sections were constructed with 3 percent cement in the base course. One of these, section 24, had an 8-inch base course, while in the remaining three, sections 7, 13, and 17, the thickness of the base was 6 inches. These four sections were continually in need of repair and, as shown in figure 7, were in the poorest condition of any group of sections on the experimental road after approximately 8 years of service. Unsatisfactory service was experienced irrespective of the type of soil from which the base was constructed, the method of curing employed, or whether the base course was 6 or 8 inches thick. Replacement of the surfacing served to improve conditions temporarily, but repeated failures in the same locations proved that the bases in these sections were structurally inadequate.

The base courses of sections 3, 8, 14, 18, and 21 contained approximately 5 percent cement. All of these sections had a designed base thickness of 6 inches except section 21, in which the nominal base thickness was 4 inches. The condition of the surface on the 5 percent soil-cement base courses after 8 years of service was better than that on the ones containing 3 percent cement. However, as shown in figure 7, a considerable portion of the area included in the 5 percent sections had failed completely, and breakage and cracking occurred in many locations. Section 21, with a 4-inch base, was the poorest of these sections. Although extensive failures occurred on fills in this section, the general distribution of failures indicates base weakness.

The full-width failure in the vicinity of station 492 in section 18 was definitely associated with fill settlement. Otherwise, this section, built on group A–5 and A–6 subgrades with group indexes generally less than 10, shows only a moderate amount of distress.

Of the remaining three 5 percent sections, section 14, built largely on group A–4 soils and cured with an emulsion cover, was in the best condition although numerous areas of isolated cracking and a few edge failures were recorded. An extensive area of breakage and cracking occurred in the vicinity of station 296 in section 3, on group A–7–6 soil. The condition of section 8, in which tar was used as a curing aid, was comparable to that of section 3, which had been allowed to cure without a protective cover.

Base courses containing approximately 7 percent cement were built on six sections, as shown in table 1 and figure 7. Extensive edge failures occurred in section 9, cured with a tar cover. Otherwise, areas of total failure were generally of smaller extent in these sections than in the sections with the same base thickness and construction variables but with 3 and 5 percent cement. Little difference could be noted in the amount of breaking and isolated cracking that occurred in the sections with 7 percent and those with 5 percent cement base courses.

The best of the 7 percent sections were section 15 (cured with an emulsion cover) and section 22, both on group A–4 soils with relatively low group indexes, and section 1 on group A–7–6 and A–7–5 soils. Section 1 was scarified to the full depth of 6 inches and recompacted about 3 weeks after the original construction. A considerable amount of cracking and breakage occurred in section 19 even though the top 2 inches of the base course were enriched with an additional 6 percent of cement following a rain during construction which interfered with proper compaction. The condition of section 4, on group A–7–5 and A–7–6 soils with high group indexes, was definitely inferior to that of other sections having bases of the same cement content but soils with lower group indexes.

Sections 2, 5, 10, 16, 20, and 23 had base courses with designed cement contents of 9 percent. The condition of these sections was generally superior to corresponding sections (those having similar base-course thicknesses, group index values, and methods of construction) having lower cement contents. Large areas of breakage and cracking in the vicinities of station 400 of section 10 and stations 547 and 560 of section 23 were associated with fill settlement.

Sections 16, 20, and 23, constructed on the more friable soils with group indexes generally less than 10, were in better condition, except in the areas of fill subsidence previously noted, than the remainder of the 9-percent sections constructed on the more plastic A–7–5 and

A–7–6 soils with group indexes generally above 10. This followed the general trend observed in the sections that have already been discussed.

Section 2, which (like section 1) was torn up and reconstructed, was superior to sections 5 and 10, which were also on soils with relatively high group indexes. The condition of the emulsion-cured section 16 was the best of any of the sections having 9 percent soil-cement bases, following another trend noted throughout the experiment.

The condition of section 6, which had a base course with a designed cement content of 11 percent, was similar to that of the adjacent section 5, with 9 percent cement and constructed on similar subgrade soils.

Fill settlement was largely responsible for the failure which occurred at the beginning of section 11. Otherwise, sections 11 and 12 were in very good condition after 8 years of service. As mentioned previously, no attempt was made to control the plasticity index or grading of the mixtures used as base courses in these sections. The principal object was to study the effect of screenings and crushed stone when used in amounts equal in cost to the cost of cement in a 6-inch base course containing 9 percent cement. Although these granular mixtures did not conform to A. A. S. H. O. specification requirements for base courses, the performance of these sections, built for the most part on group A–7–6 and A–7–5 soils having group indexes above 10, was equal to the best of the cement sections. The good performance of these sections illustrates the benefits to be derived from granular admixtures. However, it should not be inferred that the addition of crushed stone and screenings in the amounts used in this experiment will in all cases result in transforming an otherwise unsatisfactory soil into a stable base course. The construction of granular base courses without adequate control of gradation and plasticity index is not advisable and is not considered good practice.

SUMMARY

It is believed the foregoing discussion presents information that may be generally applicable to the design and construction of soil-cement bases. However, no attempt is made to draw broad conclusions since the results of one experiment of this character do not justify it. Instead, the information gathered from laboratory tests and observation of the behavior of the experimental sections of roadway is presented in summary form.

1. Portland cement was effective in changing the physical characteristics of the soils occurring on the roadway when present in the proportions used—from 3 to 11 percent.

2. In general, if the liquid limits of the raw soils were above 40 the addition of cement decreased this value, while if the liquid limits of the soils were below 40 the addition of cement resulted in a higher liquid limit (table 7). The plastic limits were with few exceptions increased by the addition of cement and in all cases a cement admixture produced a decrease in the plasticity index. Based on the average values, the addition of 11 percent cement produced a nonplastic mixture with the more plastic group A–7–6 and A–7–5 soils, while 7 percent cement was sufficient to reduce the group A–4 soils to a nonplastic condition as shown in table 7.

3. The addition of cement increased the durability of soils of the four groups encountered when subjected to either wetting and drying or freezing and thawing tests (table 3). Generally, the higher the group index, the greater the amount of cement required to produce equal durability.

4. A cement content of 3 percent did not impart to any of the soils encountered the stability required for a satisfactory base course (fig. 7). Increasing the base thickness from 6 to 8 inches, as in section 24, did not materially improve the performance of the 3 percent soil-cement mixture as a base course.

5. The performance of sections with higher cement contents in the base courses was superior to those having only 3 percent of cement. For comparable sections, i. e., those having similar base-course thicknesses, group index values, and methods of construction, there was a definite trend toward better performance with increasing cement contents (fig. 7).

6. For equal cement contents, and disregarding areas of fill subsidence, the sections constructed on the more friable soils with group indexes of 10 or less were generally in better condition after 8 years of service than those constructed on the more plastic A–7–5 and A–7–6 soils with group indexes over 10. For example, compare sections 16 and 20 with sections 2, 5, and 10, in figure 7.

7. Soil-cement base courses 4 inches thick, and having cement contents of 5, 7, or 9 percent, were not adequate on any of the soils encountered for the volume of traffic carried by this highway (fig. 7).

8. Gradual failure of soil-cement base courses 6 inches thick and with cement contents as high as 11 percent was experienced on this road on group A–7–5 and A–7–6 soils, particularly if the group index of the soil exceeded 10. For the same volume of traffic (approximately 500 vehicles per day, 15 to 20 percent of which was of the commercial type), soil-cement base courses 6 inches thick and containing 9 percent cement, appeared to be adequate on the more friable soils whose group indexes were 10 or less.

9. The use of curing aids on the soil-cement base during the hardening period appears to be desirable. In this experiment, the emulsion-cured sections were among the best of those constructed, and in every case were superior to comparable sections cured with tar. Compare sections 7, 8, 9, and 10 with sections 13, 14, 15, and 16, in figure 7.

10. The performance of sections 1 and 2 indicates that the altered physical properties of the soil produced by the addition of cement is a major contributing factor in the success of the soil-cement type base course. These sections, constructed on groups A–7–5 and A–7–6 soils with group indexes generally greater than 10, gave comparatively good results under service conditions although the hardened base course had been torn up, pulverized, and recompacted 3 weeks after it was originally constructed. Compare sections 1 and 2 with sections 4 and 5, in figure 7. After pulverizing, the cement-treated soil was very easy to reprocess with water and was similar to a friable sand-clay in workability.

11. Sections 11 and 12, whose base courses contained admixtures of granular material without the addition of cement, gave service equal to, if not better, than any of the soil-cement base-course sections.

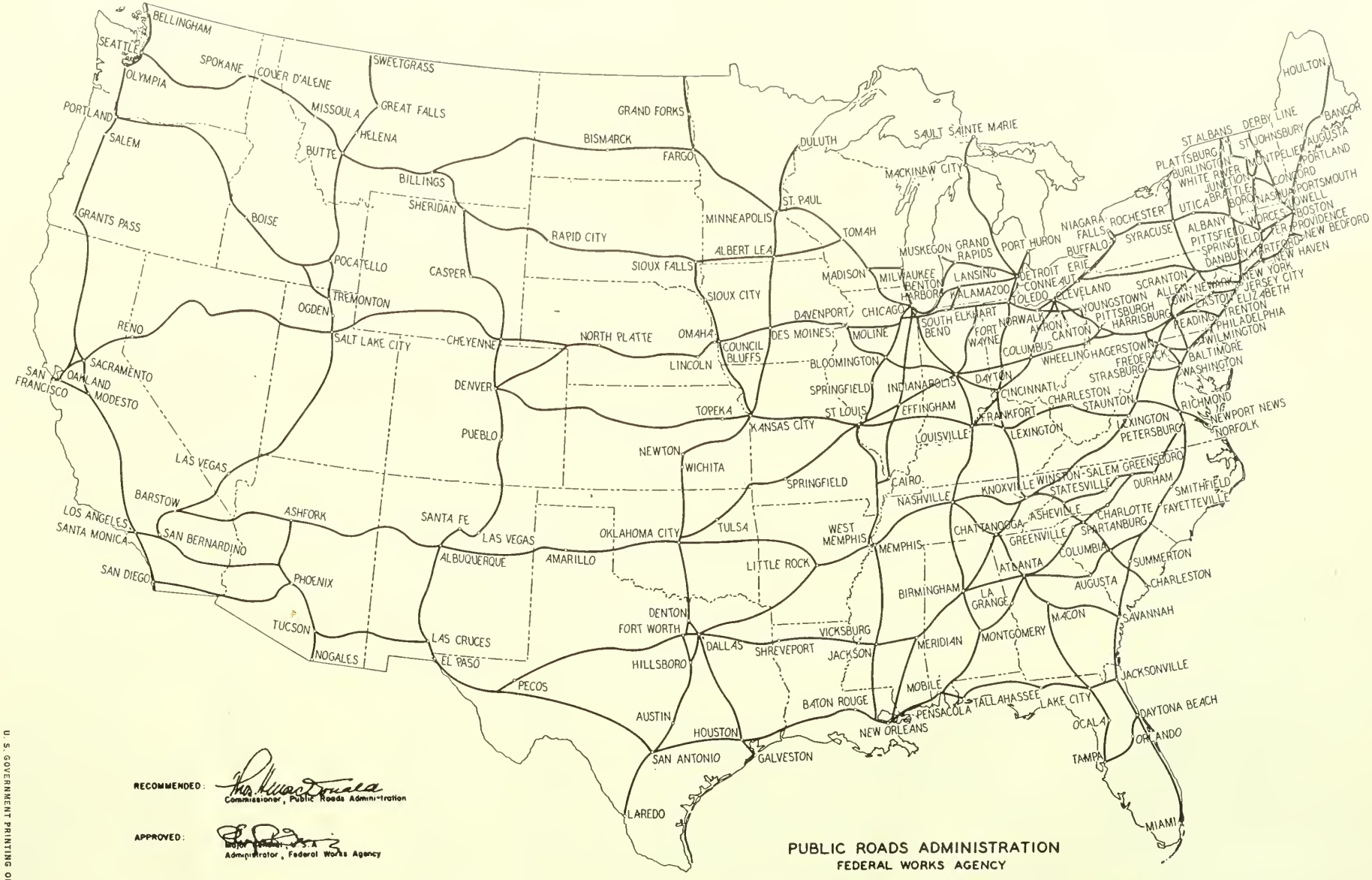
The National System of Interstate Highways

The National System of Interstate Highways (see map, p. 20), selected by the State highway departments in cooperation with the Public Roads Administration, was approved by Maj. Gen. Philip B. Fleming, Federal Works Administrator, on August 2, 1947. In recommending routes, the States were governed by the Federal-aid Highway Act of 1944 which provided for the designation of a 40,000-mile system connecting the principal centers of the country and serving the national defense.

The system as approved contains 37,681 miles of the Nation's principal highways, including 2,882 miles of urban thoroughfares. It reaches 42 State capitals and serves 182 of the 199 cities with population of 50,000 or more. All routes not already included in the Federal-aid highway system are automatically added thereto by law.

Rural sections of the interstate system comprise only 1.1 percent of all rural roads but carry 20 percent of all rural traffic. Average traffic in 1941 on the rural portions of the network was 2,693 vehicles per day, as compared with 972 on State highways.

Design standards for the system approved by the American Association of State Highway Officials call for four-lane divided highways wherever traffic is 800 vehicles or more in peak hours. Control of access, particularly in and near cities, is considered essential.



RECOMMENDED: Thos. MacDonald
Commissioner, Public Roads Administration

APPROVED: [illegible]
Administrator, Federal Works Agency

PUBLIC ROADS ADMINISTRATION
FEDERAL WORKS AGENCY

NATIONAL SYSTEM OF INTERSTATE HIGHWAYS

SELECTED BY JOINT ACTION OF THE SEVERAL STATE HIGHWAY DEPARTMENTS
AS MODIFIED AND APPROVED
BY THE ADMINISTRATOR, FEDERAL WORKS AGENCY
AUGUST 2, 1947

U. S. GOVERNMENT PRINTING OFFICE: 1947

The Hybla Valley bituminous pavement test track

A Cooperative Study of Structural Design of Nonrigid Pavements

This article serves as an introduction to the cooperative investigation of the structural design of nonrigid pavements. The participating agencies have embarked on an extensive series of experiments that will result in the collection of a large body of important data and, it is hoped, in the development of significant conclusions of great value.

The principal objectives of the investigation include development of the load-supporting values of nonrigid pavements by full-scale field tests, correlation of these data with laboratory tests to determine whether the latter can be used alone in the design of pavement thickness, and correlation of the data with in-place determinations of various values of the base-course and subgrade components.

As the investigation advances it is anticipated that a series of articles will be published to report the progress o the work, the data collected, and the conclusions drawn therefrom. This introductory article provides a description of the investigation, its purposes, and methods of procedure, and will serve as background for the articles to come.

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by A. C. BENKELMAN, Research Specialist and F. R. OLMSTEAD, Senior Soils Specialist

THIS paper is an initial progress report of the investigation of nonrigid pavement design being undertaken as a cooperative project by the Highway Research Board, the Asphalt Institute, and the Public Roads Administration. The phase of the investigation covered in this report, dealing with the construction and testing of experimental sections of pavement, was planned and begun during the war period at a time when it was difficult to achieve and maintain a normal rate of progress. While some preliminary data of a highly significant character have already been secured, the present discussion is concerned primarily with a detailed description of the project. Included is a statement of objectives, a discussion of the methods of approach, a descriptive account of the construction of the test pavement, and finally a statement dealing with the development of test apparatus and testing techniques.

Interest in the problems of structural design of the bituminous or nonrigid pavement grew enormously during the war period. To determine how thick such pavements should be to accommodate heavy airplanes, research work of a scope that would never have been considered feasible in peacetime was undertaken. Most of the work was carried on under the supervision of engineers of the War and Navy Departments. In spite of the fact that much of the work had to be planned and executed in as short a period of time as possible, a great deal of useful and pertinent information was developed. It has served to bring about a much clearer perspective of the problem and has resulted in the development of several methods of thickness design that have considerable merit.

The scope of the present cooperative investigation is sufficiently comprehensive to permit examination, study, and intercorrelation of all known theories and methods of design.

The principal objectives of the investigation include:

1. The development, by means of bearing and moving-wheel load tests on full-size pavement sections in the field, of fundamental data on the load-supporting value of nonrigid pavement surfacings of various thicknesses in combination with various base-course thicknesses and degrees of subgrade support.

2. The correlation of the field data with appropriate laboratory tests for the purpose of determining whether or not tests of the latter nature may subsequently be used by themselves as a basis for a sound method for the design of the thickness of the pavement

3. The correlation of the field data with in-place determinations of the density, moisture content, California bearing ratio, and North Dakota cone values of the base-course and subgrade components.

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Figure 1.—Aerial view of the oval test track with one tangent paved. The auxiliary test pavement is at the left of the approach road.

DESCRIPTION OF TEST PAVEMENT SECTIONS

The sections of test pavement were built outdoors with standard highway construction equipment. They are arranged in the form of an oval test track having two parallel tangents 800 feet in length connected at each end by a circular curve of 200-foot radius. The test sections themselves are confined to the tangents of the track, the connecting curves serving as a means of travel for the vehicles that will be used to apply moving loads to a portion of the width of the pavement.

The site of the experiment is near Hybla Valley, Va., on a tract of Government-owned land adjacent to U S Route 1 about 10 miles south of Washington, D. C. Views of the test area are shown in figure 1 and in the illustration at the top of page 21. The exact location for the track was selected after making a very thorough soil survey of the entire tract. While this survey revealed the presence of a fairly uniform soil formation throughout the location selected, it was decided to take every reasonable precaution to insure having a homogeneous subgrade on which to lay the test pavement. This was accomplished by building embankments of a selected soil to a height of 5 feet on the line of the tangents of the track. The equipment and methods used in the placement of the embankments are described later in this report. The moisture content of the soil was controlled within a selected range, and the material was compacted to a specified density.

The plan and profile of the completed test pavement on the north tangent of the oval track are shown in figure 2. The pavement consists of four sections, each 200 feet in length and 44 feet in width. The thickness of the base course, as indicated, ranges from 6 to 24 inches. The pavement is divided transversely into three lanes, the outer two being 16 feet in width and the center 12 feet. The thickness of the bituminous surface in the outer lanes is 3 inches and that of the center lane varies, half (100 feet) of each section being 6 inches and half 9 inches thick. Later on in this report factual data are presented relative to the design and composition of the base course and bituminous surface.

In planning the investigation as a whole, it was recognized that a considerable amount of exploratory work would have to be done in the development of test equipment and in the standardization of methods of load testing as well as of sampling and testing the different pavement components. This work necessitated the construction of a preliminary or auxiliary test pavement on which, to date, a great variety of work of an exploratory nature has been conducted.

The plan and profile of the auxiliary pavement are shown in figure 3. The total length of this pavement is 300 feet. It consists of three sections, the base and surface course thicknesses of which are indicated on the figure.

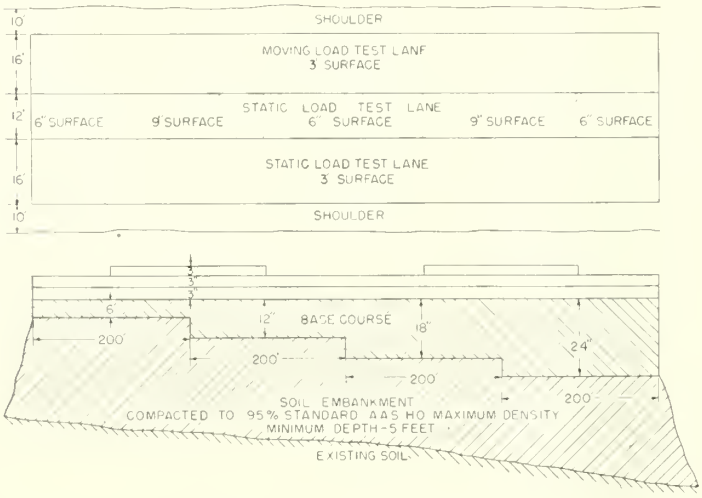
CONSTRUCTION OF TEST PAVEMENTS

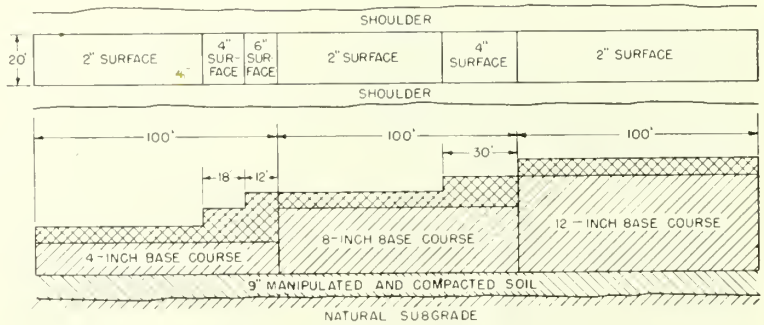
The auxiliary and oval test track pavements were built in an area containing soils of about the same physical characteristics. Tables 1 and 2 indicate typical compaction and bearing test data of the natural earth subgrade used for the auxiliary test pavement and of the selected earth borrow used for the construction of the 800-foot tangents of the oval test track. Typical test constants of the soils were as follows:

Mechanical analysis:	
passing No. 10 sieve_percent__	100
passing No. 40 sieve___do____	95
passing No. 200 sieve__do____	71
Liquid limit_____	51
Plasticity index_____	27
Shrinkage ratio_____	1. 9
Shrinkage limit_____	15
Centrifuge moisture equivalent____	35
Field moisture equivalent_____	30
Specific gravity_____	2. 76

Although the soils used for the subgrades of these test pavements were quite similar, the method of design and construction was different. The subgrade of the tangents of the oval test track (fig. 2) was built as an embankment; the subgrade of the auxiliary test pavement (fig. 3) was constructed as an all cut section in the undisturbed soil.

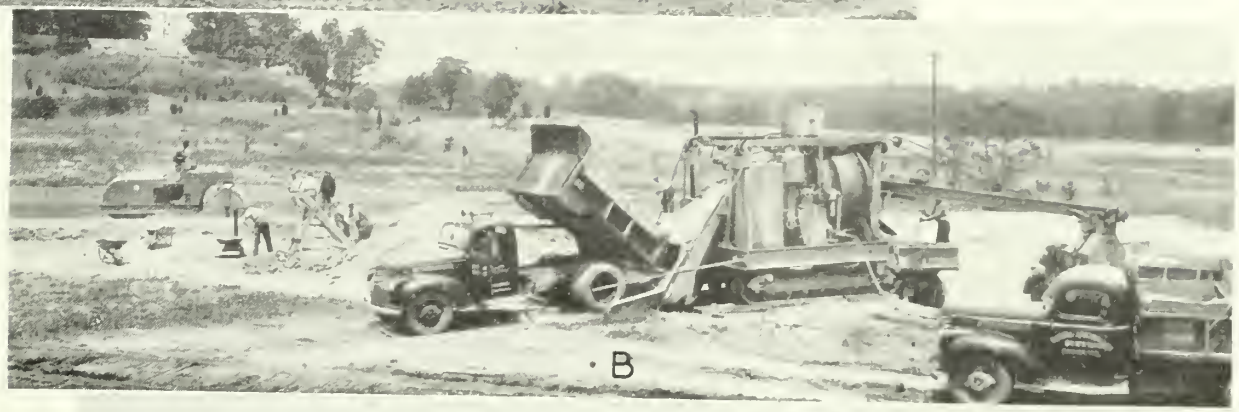
The embankments on the oval track are 5 feet high and were built with selected earth





(Left) *Figure 2.—Plan and profile, tangent section of oval test track pavement.*

(Above) *Figure 3.—Plan and profile, auxiliary test pavement.*



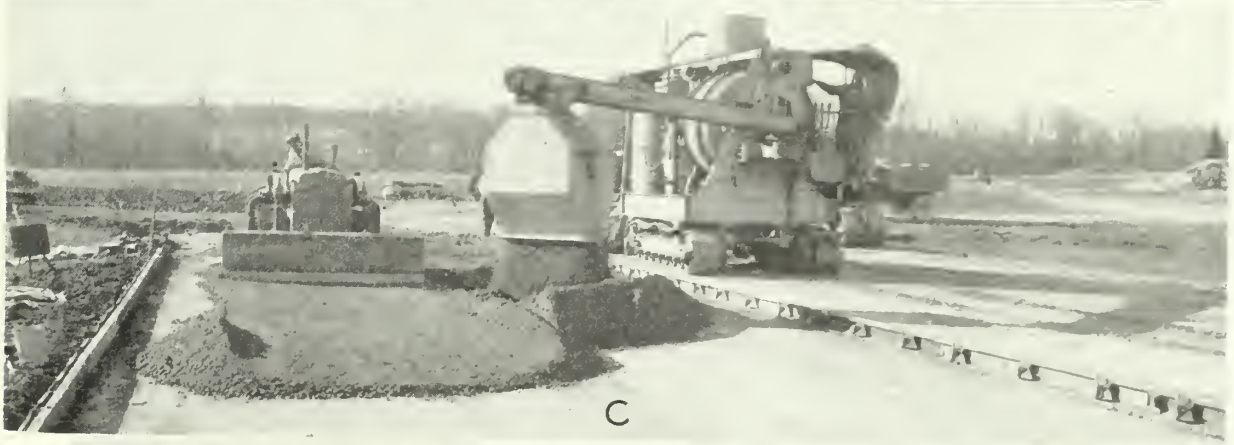


Figure 4.—Construction of the test tracks: (A) Building the oval track embankment; (B) mixing base-course materials for the auxiliary track; (C) spreading base-course materials on the oval track; (D) priming the base course; and (E) laying the bituminous surface.

Table 1.—*Typical compaction data of subgrade soil for test pavements*

Test method	Optimum moisture	Maximum dry density
	Percent	*Lb. per cu. ft.*
Standard A. A. S. H. O.	19.6	105.1
Army: [1]		
4-inch mold [2]	14.4	120.0
6-inch mold [3]	13.3	119.8

[1] Method described in U. S. Engineer Manual, March 1943.
[2] 25 blows per layer. [3] 56 blows per layer.

Table 2.—*Typical bearing test data of subgrade soil for test pavements*

Method	Bearing ratio after immersion [1]	Dry density	Moisture content [2]	Moisture content of top inch	Volume change after 96 hours
	Percent	*Lb. per cu. ft.*	*Percent*	*Percent*	*Percent*
Army	2.0	106.3	21.8	[3] 30.0	2.0
California	2.0	112.1	18.4	[3] 28.7	13.5

[1] Bearing ratio at 0.1-inch penetration.
[2] Based on dry weight. [3] 10-pound surcharge.

Table 3.—*Design characteristics of stabilized aggregate base courses*

	Specification limits [1]	Auxiliary pavement base course [2]	Oval test track base course [2]
Percent passing sieve size:			
1½ inch	100	100	100
1 inch	100	99	95
¾ inch	70–100	90	85
⅜ inch	50–80	67	67
No. 4	35–85	56	55
No. 10	25–50	47	46
No. 40	15–30	24	20
No. 200	5–15	9.5	6
Liquid limit	25 max	17	17
Plasticity index	6 max	4	3
In-place density [3]		130	136
Number tests averaged		13	70

[1] A. A. S. H. O. stabilized aggregate base-course specification B–1.
[2] Average of all tests.
[3] Dry density in pounds per cubic foot.

borrow compacted in 4-inch layers to at least 95 percent of standard A. A. S. H. O. maximum density at moisture contents less than 2 percent above the optimum. The subgrade of the auxiliary test pavement consisted of the natural subgrade with the upper 9-inch layer manipulated and compacted to the same minimum compaction requirements specified for the embankments of the oval track.

The field control of the subgrade compaction consisted of making moisture tests of soils before rolling, and in-place density and moisture tests after compaction. Tests were made for each 400 cubic yards of material compacted; and whenever areas were found to have less than the required density they were reworked and recompacted until satisfactory in-place densities were obtained.

In general, the moisture content of the natural soils was slightly above that required for satisfactory compaction. The excess soil moisture was removed during the manipulation operation before starting the compaction. The placement, manipulation, and compaction operations were planned so that a 4-inch layer of soil could be placed over the entire length and width of the 800-foot tangent during each day of construction.

Several precautions were taken to avoid construction delays induced by adverse weather conditions. Before completing each day's work the surface of the embankment was shaped with motor-patrol graders and rolled with a pneumatic-tire roller to minimize any ponding of water on the embankment in the event of a rain. Likewise, the selected earth borrow area was graded and maintained free from surface irregularities to avoid ponding of water. The selected earth borrow was removed in shallow layers by pan scrapers along the line of the prevailing ground slopes, and drainage ditches were maintained to insure adequate drainage of the borrow area.

After the subgrades for both test pavements were completed they were shaped and rerolled to the alinement and cross sections designated by the plans. During the course of the construction of these test pavements it was found necessary to finish the auxiliary test pavement prior to the completion of the oval test track. Certain experimental design data on pavement thickness, compaction of the stabilized aggregate base course, and probable performance of the bituminous wearing course under static loading were needed to plan the final design of the base and wearing courses for the 800-foot tangents of the oval test track. The auxiliary test pavement designed for this preliminary study was constructed with 4-, 8-, and 12-inch base courses with several different thicknesses of bituminous wearing courses, as shown in figure 3.

The stabilized aggregate mixture used for the base course of the auxiliary test pavement was designed to conform with the A. A. S. H. O. B–1 grading specifications for stabilized aggregates. In the case of the oval test track, however, this specification was modified to permit the use of commercial coarse aggregate containing a slight amount of oversize material (passing 1½-inch sieve, retained on 1-inch sieve). Table 3 indicates the design characteristics of the stabilized aggregate base-course mixtures for both test pavements.

These stabilized aggregate mixtures were prepared by blending commercial coarse and fine aggregates on a weight basis at the aggregate plant. After the preweighed batches of aggregates were hauled to the project site, pulverized binder soil was added to each batch from a proportioning bin and the materials were uniformly mixed in a concrete paving mixer. The moisture necessary for proper compaction was added during the mixing operations. In the construction of the auxiliary test pavement, the mixer was operated at a fixed location and the mixed material was hauled to the site in trucks. On the oval test track, the mixer operated and discharged directly on the embankment subgrade.

The stabilized aggregate base courses were built using standard construction procedures. The material was spread in 3-inch layers and compacted by means of smooth-face and pneumatic-tire rollers until the required density was obtained.

The essential differences between the base courses of the auxiliary test pavement and the oval test track were in the thickness and compaction requirements. The base course of the auxiliary test pavement was compacted to an in-place dry density of 130 pounds per cubic foot and was built in three thicknesses—4, 8, and 12 inches. The base course for the north 800-foot tangent of the oval test track was compacted to an in-place dry density of 136 pounds per cubic foot and was constructed in four thicknesses—6, 12, 18, and 24 inches.

The asphaltic prime coat which was applied to the surface of the completed base course was the same for both test pavements. MC–1 was applied at a rate of 0.25 gallon per square yard. The bituminous concrete pavement

Table 4.—*Bituminous mixture specifications for the test pavements*

Constituent	Auxiliary test pavement A–2 [1]	Oval test track pavement	
		Binder A–2–a II [1]	Surface A–2–a IV [1]
MINERAL MATTER:			
Coarse aggregate: [2]	*Percent*	*Percent*	*Percent*
Passing 1½-in. sieve, retained on 1-in.	0– 5		
Passing 1½-in. sieve, retained on ¾-in.		14–48	
Passing 1-in. sieve, retained on ¾-in.	15–25		
Passing ¾-in. sieve, retained on ⅜-in.		3–45	18–50
Passing ¾-in. sieve, retained on No. 4	20–35		
Passing ⅜-in. sieve, retained on No. 4			3–36
Passing No. 4 sieve, retained on No. 10	5–15	5–15	9–22
Subtotal	60–70	55–75	50–65
Fine aggregate: [3]			
Passing No. 10 sieve, retained on No. 40		3–21	5–22
Passing No. 40 sieve, retained on No. 80		6–25	9–27
Passing No. 80 sieve, retained on No. 200		3–16	5–18
Passing No. 10 sieve, retained on No. 200	25–35		
Mineral filler passing No. 200 sieve	4– 6	4– 6	5– 8
Subtotal	30–40	25–45	35–50
Total mineral matter	100	100	100
TOTAL MIX:			
Mineral aggregate	92–95	93–95	92–94
Asphaltic cement [4]	5– 8	5– 7	6– 8
Total	100	100	100

[1] Asphalt Institute dense graded aggregate specification, hot-mix type.
[2] Crushed limestone.
[3] Natural river sand.
[4] Asphaltic cement, 85–100 penetration, Federal specification SS–A–706–b.

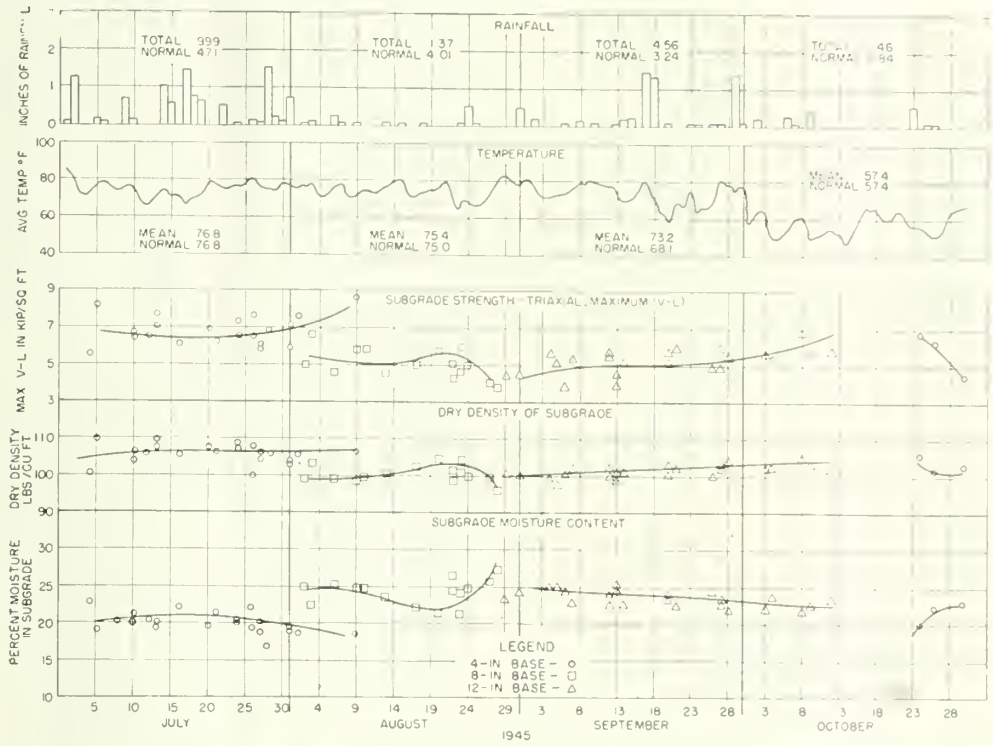


Figure 5.—Relation between weather conditions and strength, density, and moisture content of subgrade under base courses.

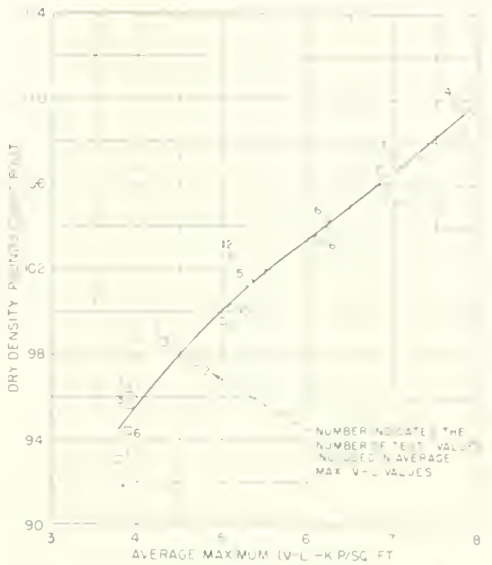


Figure 6.—Relation of maximum V—L (triaxial compression) to density.

was placed after the prime coat had been thoroughly cured.

The bituminous concrete paving mixtures used on the auxiliary pavement and on the oval track were of the hot plant-mixed type conforming to the design requirements usually specified for highway construction. Table 4 gives the specification requirements of the two paving mixtures. The placement and compaction operations were similar to those recommended for highway construction of this pavement type. Typical views of the construction of the auxiliary and oval track pavements are shown in figure 4.

SUBGRADE STUDIES OF AUXILIARY TEST PAVEMENT

A limited study of the behavior of the subgrade beneath the base course of the auxiliary test pavement was made to investigate the effect of the climatic factors—rainfall and temperature—upon subgrade strength (max. $V-L$),[1] density, and moisture content. The initial work was confined for the most part to study of the behavior of the 9-inch compacted layer of soil directly beneath the base course. However, at periodic intervals the moisture gradient was determined for the natural soil to a depth of approximately 36 inches, in order to obtain relative data for comparison of moisture movements at various elevations within the zone that might be influenced by the proposed plate loading of the test pavement.

The variations in strength, density, and moisture content of the 9-inch compacted subgrade layer are shown in figure 5. A comparison of these data show the following trends:

1. The strength (max. $V-L$) varies directly as the density and inversely as the moisture content of the soil.

2. The moisture content gradually increases with time and with the seasonal drop in average daily temperature.

3. Minor variations in strength, density, and moisture content appear to be related to the monthly fluctuations in the climatic factors—temperature and rainfall.

The general relations between these variables were obtained from the averaging of a number of individual test results. Figures 6 and 7 indicate the relations of the average strength (max. $V-L$) to the dry density and to the moisture content of the compacted 9-inch subgrade layer.

The relations of average modulus of elasticity of the subgrade soil, calculated from the triaxial compression test data, to moisture content and to density of the subgrade are shown in figure 8. The modulus varies directly as the density and inversely as the moisture content of the subgrade.

The average seasonal strength (max. $V-L$), density, and moisture content of the soil in the 9-inch compacted layer for several seasons of the year are tabulated in table 5. A study of these data show that the strength of the soil varies with the seasons, being lowest during the winter and highest in the summer. The strength varies inversely and the density directly with the moisture content of the soil. These seasonal variations emphasize the need of planning comparative pavement-evaluation

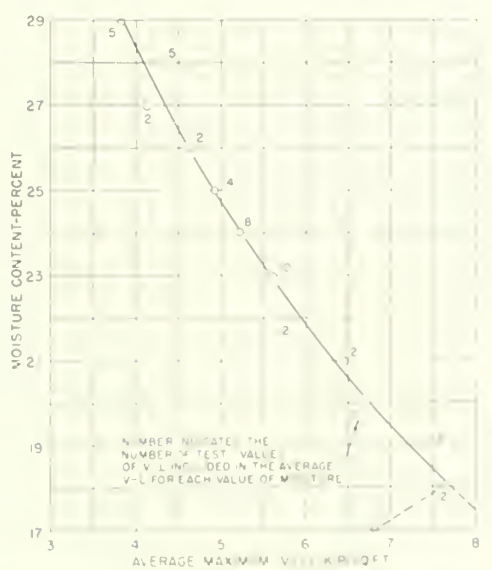


Figure 7.—Relation of maximum V—L (triaxial compression) to moisture content.

[1] The subgrade strength (max. $V-L$) as used in this study refers to the maximum difference between the vertical and lateral pressures as measured by the triaxial compression test

Table 5.—*Seasonal variations in subgrade moisture, density, and strength under the 4-inch base of the auxiliary test pavement* [1]

Type of test	Date of tests, 1945–46—				
	July 10–15	Oct. 24–25	Nov. 23–27	Jan. 10–11	Mar. 4
Number of tests averaged	5	3	2	2	4
Average moisture, percent	20.0	21.7	28.3	29.0	28.5
Average dry density, lb. per cu. ft.	105.6	103.3	95.0	93.5	95.5
Average maximum $V-L$, kips per sq. ft.	7.0	5.8	3.5	3.4	4.7
Average saturation, percent	89	91	98	96	98

[1] All tests were sampled from an area approximately 55 feet in length.

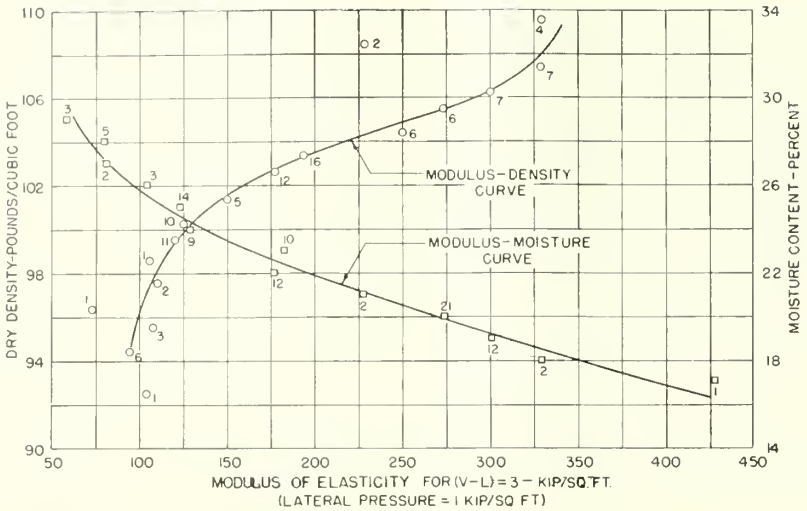


Figure 8.—Relation of modulus of elasticity to moisture content and density of the auxiliary test track subgrade.

tests so that they can be made during periods when the climatic factors are uniform. Adverse changes in climate affect the strength of both the subgrade and the pavement. Both of these strength values must be considered in the evaluation of pavement test data.

The periodic variations in subgrade moisture content at several elevations below the three thicknesses of the base course of the auxiliary test pavement are shown in table 6. A study of these data shows that there appear to be both horizontal and vertical fluctuations in the subgrade moisture content found beneath the base course. Reversals in the moisture contents at various elevations seem to occur for no apparent reason. Perhaps minor variations in texture or structure of the undisturbed soil, in the form of thin lenses of silt and fine sand or seepage planes, may be one of the causes for the erratic fluctuations in the subgrade moisture content beneath the test sections.

It was noticed during field examinations of the general area that in some instances, during the spring season, auger holes gradually filled with free water even though no rain fell during the period between making the auger holes and the observation of the free water. In the area in which the auxiliary pavement was built, there was a definite relation between periods of heavy rainfall and the elevation of free water in the test pits dug in the construction area.

Subgrade conditions similar to those found in the area under the auxiliary test pavement should not occur beneath the 800-foot tangents of the oval test track because the subgrade of the latter is a fill section, composed of soils that were uniformly mixed and compacted to a depth of 5 feet to minimize moisture variations resulting from the change in texture or structure.

A 16-point automatic temperature recorder will be installed in the subgrade, base course, and bituminous pavement of the oval test track to measure the variations in temperature at various elevations below the surface of the pavement. These continuous records of temperature will be used to study the insulation effect of the various thicknesses of pavement as well as to furnish temperature data for use in the correlation of the pavement evaluation work.

LOAD-BEARING SURFACES

In connection with the load testing of nonrigid pavements it was recognized that the use of rigid plates would cause stresses in the pavement structure radically different in character from those produced by pneumatic tires. The reason for this is that the material beneath a rigid plate must displace equally at all points even though the resistance offered to displacement is not uniform. In a cohesive material, such as a bituminous pavement, the resistance to displacement will be greater at the boundaries of the load-bearing surfaces and less in the central portion. Consequently the contact pressure that will develop beneath the peripheral area of a rigid plate as it is forced into a bituminous pavement may exceed greatly that beneath the central portion of the plate. Under a pneumatic tire the situation is quite different. Here, because the tire can undergo a change in shape without a significant change in its pressure-distribution pattern, the material being loaded will tend to deform in a natural manner.

The use of tires themselves as static load-bearing surfaces was not considered feasible (1) because the contact area does not remain constant but increases with load, (2) because of the physical difficulty of measuring the deflection of the material being tested, and (3) because tests are to be made on the base-course and subgrade components of the pavement, and the opening to accommodate the tire would have to be larger and of a more irregular shape than desired.

Thus, one of the important instrumental problems was the development of a type of load-bearing surface that would simulate the action of a tire and yet have none of its undesirable physical characteristics.

In the development work on this problem it was necessary to study the pressure distribution characteristics of a variety of materials. A special pressure-indicating apparatus was constructed with which the contact pressure at any point beneath a particular loaded material could be determined.

The fact that sponge-rubber mats had been used to reduce high peripheral pressures beneath rigid plates in concrete-pavement loading tests prompted a series of initial studies with this material. The effect of the type and thickness of the rubber cushion, both when confined and when nonconfined laterally, was studied. It was found that in either case the pattern of pressure distribution was extremely irregular. When unconfined, the rubber cushion displaced laterally under load, thus reducing the normal intensity of the pressure beneath the peripheral area and increasing the pressure beneath the central area of contact. When confined, an edge or perimeter effect was created resulting in a high concentration of pressure at the outer limits of the area of contact.

Table 6.—*Variations in subgrade soil moisture content (in percent) at several depths below the surface of the subgrade of the auxiliary test pavement, at various times of the year*

Location of sampling area	Depth below bottom of base course	Moisture content for date sampled (1945–46)										Moisture range			Average moisture content
		Aug. 30	Sept. 27	Oct. 16	Oct. 31	Jan. 11	Feb. 20	Mar. 5	Mar. 20	Apr. 2	Apr. 16	Minimum	Maximum	Difference	
	Inches														
Natural subgrade [1]	3	17.7	17.9	17.7	18.0	22.5	20.5	26.9	22.0	22.2	20.7	17.7	26.9	9.2	20.6
4-inch base section	1	20.9	19.7	20.6	21.3	-------	22.5	-------	22.3	21.7	23.0	19.7	23.0	3.3	21.5
8-inch base section	1	27.1	26.6	27.8	26.8	-------	26.6	-------	28.8	27.0	24.8	24.8	28.8	4.0	26.9
12-inch base section	1	23.2	24.2	24.5	22.7	-------	24.6	-------	24.3	24.7	20.1	20.1	24.7	4.6	23.5
Natural subgrade [1]	14	25.8	35.1	35.3	38.5	35.3	-------	22.1	21.9	24.3	24.8	21.9	38.5	16.6	30.4
4-inch base section	10	23.1	23.5	25.0	22.9	-------	23.3	28.2	29.2	24.3	-------	22.9	29.2	6.3	24.9
8-inch base section	10	19.4	25.0	22.6	21.7	-------	25.6	25.2	24.6	25.9	22.7	19.4	25.9	6.5	23.6
12-inch base section	10	23.9	24.4	24.7	25.9	-------	22.9	20.6	26.3	28.5	18.2	18.2	26.3	8.1	23.9
Natural subgrade [1]	26	31.9	31.7	30.1	31.0	35.7	36.6	33.5	36.0	30.7	32.3	30.1	35.7	5.6	33.0
4-inch base section	22	26.8	26.3	25.0	24.7	25.6	24.0	22.2	27.3	-------	26.4	22.2	27.3	5.1	25.4
8-inch base section	22	22.8	24.0	24.4	21.8	25.6	25.4	23.8	23.9	27.2	24.1	21.8	27.2	5.4	24.4
12-inch base section	22	23.5	26.2	24.6	24.5	-------	29.2	25.7	27.0	25.0	23.8	23.5	29.2	5.7	26.6

[1] Natural subgrade refers to an undisturbed subgrade location adjacent to the auxiliary test pavement.

Considerable time was spent in a study of the pressure distribution characteristics of a specially built rubber bag. This was composed of gum rubber molded in a cylindrical shape, and had a vertical hole through its center through which deflection measurements of the material under test could be made. This rubber bag was tested in a number of ways, with and without lateral confinement, both when inflated with air and when filled with water. The distribution of pressure beneath the bag was erratic. When confined laterally and overinflated with either air or water, the pressure was concentrated largely over an annular section between the perimeter and center opening of the bag. When confined laterally and underinflated, a high concentration of pressure developed beneath the side wall and around the center opening. When unconfined, the bag compressed vertically under load to the extent of creating abnormal pressure beneath the side walls and around the center opening. There appeared to be no way in which the bag could be used so that the carcass or vertical walls of the unit did not adversely influence the pattern of pressure distribution.

Many other types of material were studied from the standpoint of their potential ability to transmit load and give the desired pattern of pressure distribution. Among other things, the pressure-transmitting characteristics of thin rubber membranes were investigated in considerable detail. The fact that the pressure transmitted by such a material was invariably found to be equal to the pressure imposed upon it led to the decision that the ultimate design of the bearing surface should embody the use of such an elastic membrane as the primary pressure-transmitting medium. The problem resolved itself into one of determining how to attach the membrane to a chamber or cell so that when air under pressure is introduced (1) the membrane would not blow out, (2) the contact area would not vary to any appreciable extent with load, and (3) the intensity of the transmitted pressure over the entire facial area of the bearing surface would be reasonably uniform.

Three types of bearing cells employing an elastic membrane were developed. They are shown schematically in figure 9. The cell on the left is simply an air chamber with intact side walls. The cell in the center has two cylindrical sections, the lower telescoping into the upper. The cell on the right is similar to that on the left except that a section of flexible metal bellows is installed in the cylindrical section. The performance characteristics of all three cells were studied both with the elastic diaphragm attached to the inside and to the outside of their respective base sections.

From a standpoint of the forces of action and reaction all three cells are basically the same when the diaphragm is on the inside. A given internal air pressure will produce an upward reaction, R, equal to the area of the diaphragm times the prevailing pressure: $R=A\times P$. To develop the same unit intensity of pressure beneath the rim or end projection of the cylinder as that acting through the diaphragm, a supplemental load, L, can

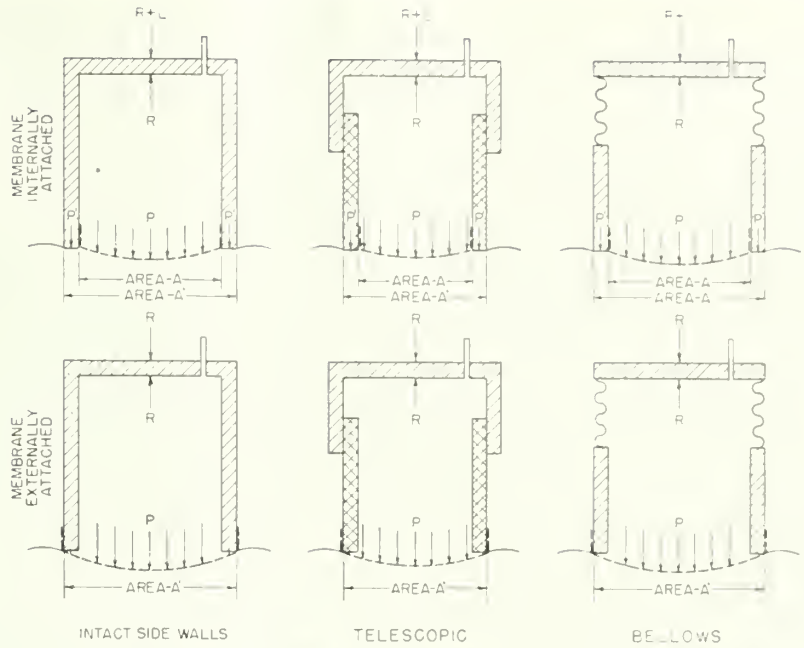


Figure 9.—Schematic cross sections of the flexible-surface load-bearing cells.

be applied by external means to the top of the cell, of a magnitude such that L equals rim area times inflation pressure.

Thus it appears that any one of the three cells could be used in such a way that the contact pressure would be uniform over its entire facial area—that is, beneath both the diaphragm and the end projection of the rim section. In this case the rim section would be a part of the bearing surface and would constitute its peripheral area. However, in tests with the cell having intact side walls, extreme difficulty was encountered in manually controlling its operation so that load L at all times was in balance with load R. There appeared to be no way, mechanically or automatically, of maintaining the desired balance between the two components of the load as the internal air pressure is increased or decreased during the progress of a test.

The telescopic cell was developed in an attempt to overcome the operational difficulties of the cell with intact side walls. It was reasoned that in a cell of telescopic design the internal air pressure would bear against the upper projection of the lower section and that this pressure, less the effects of friction that may be present or may develop as the sections move with respect to one another, would be transmitted downward to the material under test. In other words, the pressure transmitted automatically by the end projection of the rim section would be approximately equal to that of the internally existing air pressure. In order to prevent undue leakage of air from the space between the sections, however, it was necessary to use a leather gasket seal that had a relatively high coefficient of friction. This seal served to reduce the pressure transmitted by the rim about 30 percent when the sections were in the process of opening, and to increase this pressure about the same amount when the sections were closing.

It is possible that a bearing surface of the telescopic type, having the diaphragm attached to the inside of the base section, could be used in a manner that would tend to promote continuous and progressive opening of the movable sections during the course of a test. This would result in a reasonably constant deficiency of pressure beneath the rim section, and such a situation might not be considered objectionable if the ratio of the rim area to the diaphragm area is low. However, if it is considered essential to have the same intensity of pressure transmitted through the rim section as that acting through the diaphragm, the total reaction load must be controlled or made to equal the total facial area of the cell times the internally existing air pressure. In contrast to a cell having intact side walls, this can be accomplished without serious danger of overloading or underloading the rim section.

The bearing cell shown at the right in figure 9 was conceived in an effort to overcome the undesirable features of the intact side wall cell and the telescopic cell. It was reasoned that the bellows convolutions installed in the cylindrical section would allow a certain measure of vertical movement without altering the distribution of pressure over its contact face. A considerable amount of work has been done with a cell of this type. While the bellows section, designed to withstand an internal pressure of 100 pounds per square inch, exhibited a higher spring rate than desired, it did serve to impart a much greater degree of vertical flexibility to the cell than when the side walls were intact. Because of this vertical flexibility, the load may be applied and released more uniformly and with less likelihood of rupturing the membrane than was experienced with the rigid-walled cell.

The foregoing discussion described the three flexible bearing surface cells with the dia-

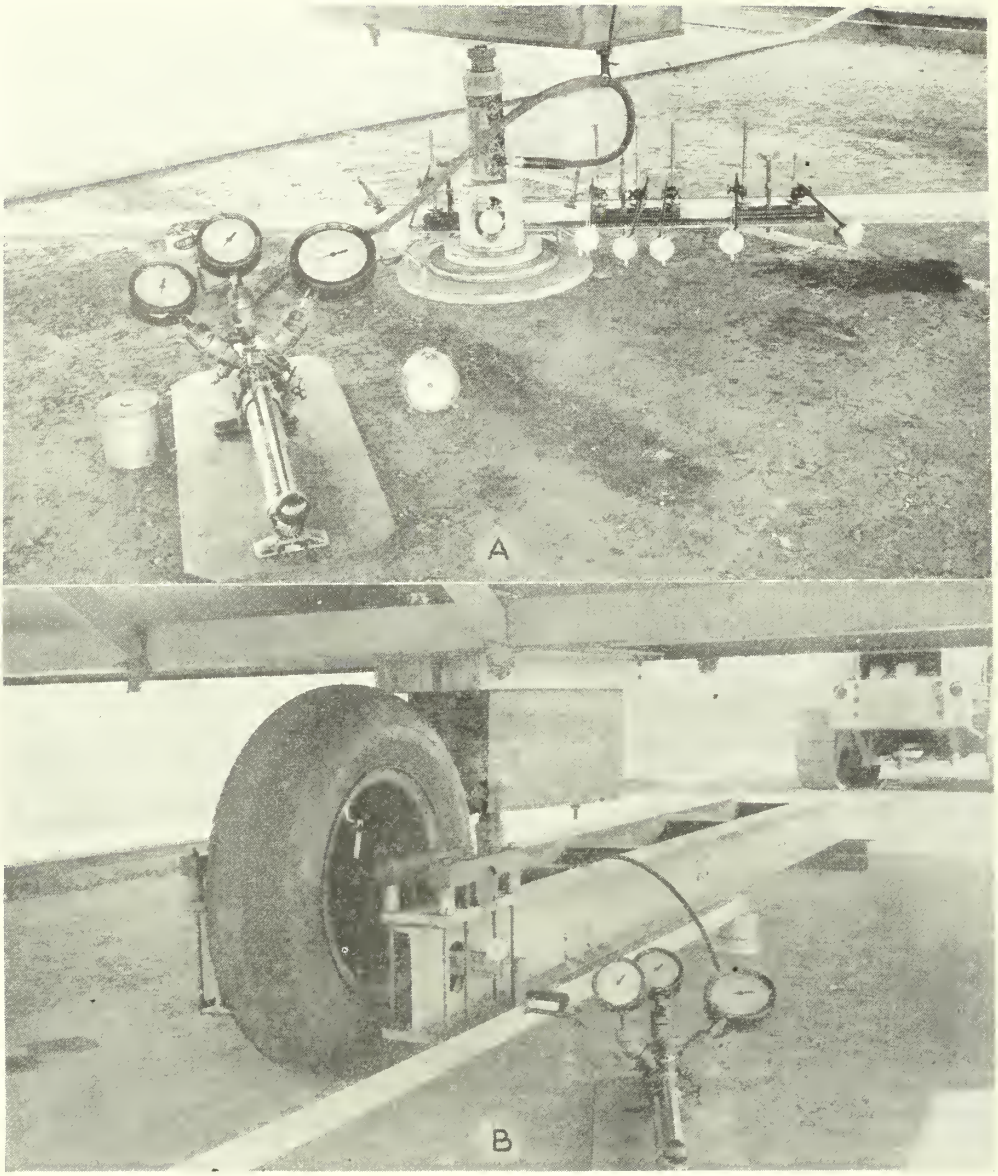


Figure 10.—Load-test assemblies using (A) *rigid plates and* (B) *a pneumatic tire.*

phragm attached to the inside of the lower cylindrical section. In some of the preliminary tests with these cells it was found that the peripheral rim section left what appeared to be an abnormal cut or sharp impression in the material tested. This was in marked contrast to the impression left by a loaded tire on the same material. Here the impression around the edge of the imprint area was relatively smooth.

As a result of these observations the possibility of attaching the diaphragm to the outside rather than to the inside of the unit was considered. It was realized that in this event the contact face of the unit would vary depending upon the magnitude of the applied load, the carcass stiffness of the diaphragm, and the amount of clearance between the end projection of the rim and the diaphragm or material on which it rests. As a result it is not possible to formulate the forces of action and reaction to the same degree of accuracy as when the diaphragm is attached to the inside. It has been found, however, by controlling the amount of clearance between the rim and diaphragm, that the variations in size of contact area are of negligible proportions. In tests with this type of diaphragm attachment the tendency toward the development of a shearing or cutting action at the perimeter of the contact area is greatly reduced.

LOAD-TESTING PROCEDURES

One of the purposes of the auxiliary test pavement was to serve in the development of a procedure for conducting the load-bearing tests on the oval track pavement—a procedure which could be relied upon to give the best possible results. Methods for making such tests have never been standardized, particularly those having to do with the load-supporting capacity of pavements.

Three procedures of load testing using rigid plates were studied. The first involved the application of five or more load increments, each increment being applied and released five times. Each increment produced a net deflection of the medium under test of 0.1 inch—the five increments thus producing a gross deflection of 0.5 inch. In tests using this procedure the applied load was not released nor was the released load reapplied until the rate of vertical movement had slowed down to 0.002 inch per minute. Tests were made according to this procedure on the pavement surface, base course, and subgrade of the three sections of the auxiliary pavement (see fig. 3) using plates of 9- and 18-inch diameter. Duplicate tests were made in all cases, and where the results of the two tests were not in reasonably good agreement a third test was run.

The second procedure differed from the first in that the load was applied rapidly and without interruption until visible rupture of the material occurred or until its resistance was overcome. The application of the load was such as to produce a reasonably constant rate of deflection of 1.2 inch per minute. Tests were conducted according to this procedure on the pavement surface, base course, and subgrade of the auxiliary pavement sections with four rigid plates of 6-, 9-, 12-, and 18-inch diameter. In addition a special series of tests was made, using this procedure, in an effort to develop some preliminary information relative to the effect of the temperature of the bituminous surface upon the load-supporting capacity of the pavement structure. Tests were run with the same group of rigid plates upon the 2-, 4-, and 6-inch thicknesses of bituminous surface over the 4-inch base section of pavement when the temperature of the surface was about 68°, 85°, and 105° F.

The third procedure was that of repetitional loading in which from 15 to 100 load applications were applied and released. Tests were made on the top of the base of all three auxiliary pavement sections with the 9- and 18-inch diameter plates. The series of tests was planned specifically for the purpose of studying the elastic action of the base course-subgrade component of the pavement.

In addition to the various tests described above a great number of tests were conducted upon the sections of the auxiliary pavement in connection with the problem of the development of a flexible type of bearing surface. The several load-test assemblies are shown in figures 10 and 11. A view of a rigid plate assembly appears in figure 10*A*. The plates are arranged in typical pyramid fashion to insure rigidity. Vertical movement readings are made with dial gages bearing against the center and edges of the base plate and against the surface of the pavement at distances of 3, 6, 12, and 24 inches from the plate. Load is applied by means of a retractable hydraulic jack. In this as well as in all the other load assemblies, an electric buzzer is clamped to the dial-supporting beam for the purpose of freeing the dial stems when readings are made. The effect is similar to that of tapping a gage to make sure that the needle is not stuck. It has been found that the use of such a buzzer not only serves to facilitate reading the gages but tends to give more accurate results.

Figure 11*A* shows a load assembly employing the telescopic type of bearing surface described previously. The deflection of the pavement surface in this case is measured at the center of the unit on the top of a rod ex-

tending downward through the cell to the diaphragm. The deflection of the rim of the cell or the material beneath the end projection of the rim is measured by means of three dial gages operating against brackets attached to the rim. The air pressure in the cell is controlled by means of the pressure regulating valve assembly shown. The hydraulic jack superimposed on the cell is used to determine or control the reaction load. In figure 11*B* another load-test assembly is shown, employing the bellows type of bearing surface. This cell is operated and the deflections measured in much the same manner as described for the telescopic cell.

In figure 10*B* still another load assembly is shown, utilizing a pneumatic tire as the load-bearing surface. Here the vertical movement of the pavement surface beneath the center of the tire contact area is measured by means of a specially designed deflection rod unit that is installed in the tire. Lack of clearance beneath the water tank that was used on the auxiliary test track to provide the necessary load reactions made it necessary to mount the tire on the end of the beam assembly shown and apply the load in the manner indicated.

In load testing the pavements and base courses of the oval test track a 25-ton trailer, loaded with concrete or steel blocks, is being used. The weight is concentrated on the bearing surface by means of a hydraulic jack, as shown in the cover illustration.

SUMMARY

A great deal of pertinent and useful information has been obtained from the tests made to date on the sections of the auxiliary pavement. This information has been particularly helpful in the formulation of the testing program for the sections of pavement on the oval test track.

The investigation has been planned in such a way that it should be possible to compare and correlate all existing proposed methods of thickness design. Thus in the course of the investigation such tests as the North Dakota cone test, the California bearing ratio test, and the triaxial compression test must be made on certain of the component materials of the pavement at each location where a bearing test is made. In addition, samples of these components will be obtained for moisture content and density determinations.

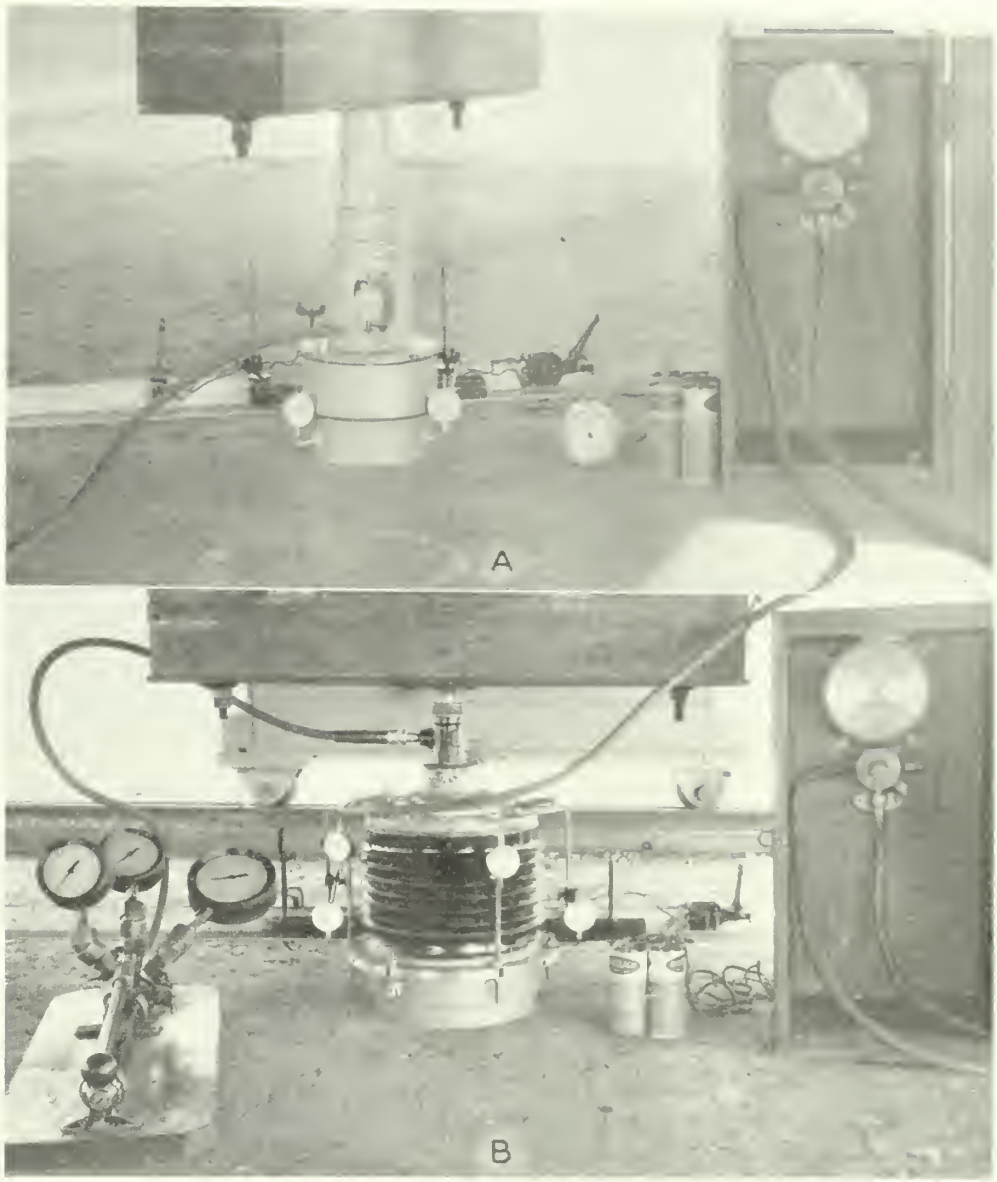
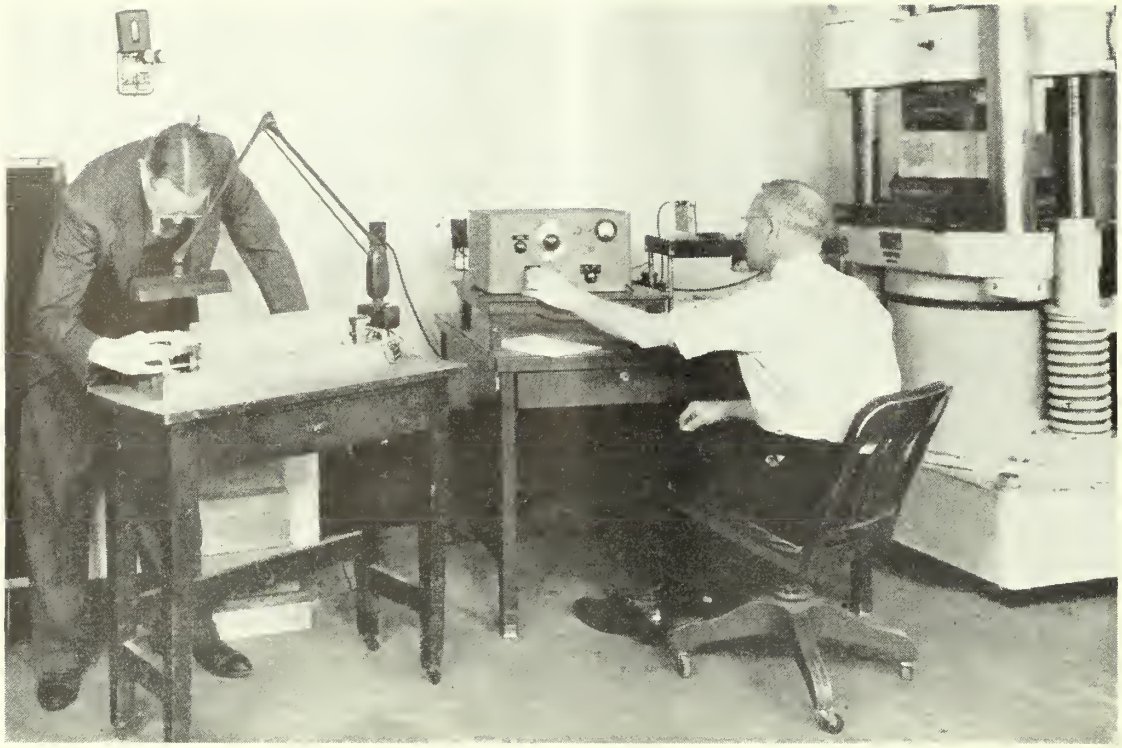


Figure 11.—Load-test assemblies using (A) *the telescopic cell and* (B) *the bellows cell.*

The completion of the planned program of tests should throw considerable additional light on many questions which are as yet, to a great extent, unanswered.



Measuring changes in volume and sonic modulus of elasticity of frozen and thawed concrete beams

The Effect on Properties of Concrete of Natural and Portland Cement Blends[1]

Confirming the results of earlier studies, this investigation of concretes made with blends of natural and portland cements shows that such concretes, because they entrain air, have increased resistance to freezing and thawing. Offset against this gain is a somewhat lessened strength. The investigation found no properties in the natural cements, other than their air-entraining ability, that influenced the increased resistance to freezing and thawing.

In plain concrete slabs as constructed in the field, the top of the slab usually has lower resistance to freezing and thawing than the bottom. This investigation indicates that air entrainment improves the concrete throughout the slab with regard to resistance to freezing and thawing and tends to make it more uniform.

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by A. G. TIMMS, Senior Materials Engineer, W. E. GRIEB, Associate Highway Engineer, and GEORGE WERNER, Associate Materials Engineer

AN INVESTIGATION of the use of blends of natural and portland cement for improving the quality of concrete was reported in 1938 by the Public Roads Administration.[2] That investigation, prompted by the increasing use of blends in the field, was limited to the use of one natural cement in the case of the laboratory-fabricated specimens and to two natural cements in the case of cores drilled from pavements and tested in the laboratory.

The present report summarizes the results of a considerably expanded laboratory investigation begun in January 1942 that involved the use of seven natural cements and one slag cement, each of which was blended with each of three plain or nonair-entraining portland cements. In addition, comparable tests were made with the three plain portland cements as well as with the air-entraining counterparts of two of them. Some of the natural cements contained sufficient air-entraining material to produce air-entraining concrete, while others produced little or no additional air.

For convenience in conducting this investigation the tests were divided into three series, each designed to study certain properties of concrete. Series 1 was planned to study the flexural and compressive age-strength relations; series 2[3] was designed to investigate drying shrinkage and resistance to freezing and thawing in water, and to study the effect of initial curing and an interruption in the freezing and thawing cycle; and series 3 covered an investigation of the effect of segregation of the constituent materials due to manipulation of the surface in the finishing operations in laboratory-mixed concrete placed to simulate pavement construction practice.

CONCLUSIONS

In this investigation the plain portland cements, the air-entraining portland cements, and the natural cement blends entrained air in volumes of from 1.3 to 5.3 percent—all well

[1] Presented at the fiftieth annual meeting of the American Society for Testing Materials, June 17, 1947, at Atlantic City, N. J.

[2] *The Effect of Using a Blend of Portland and Natural Cement on the Physical Properties of Mortar and Concrete*, by W. F. Kellermann and D. G. Runner, PUBLIC ROADS, vol. 19, No. 8, October 1938, p. 153, *also* Proceedings of the American Society for Testing Materials, vol. 38, Part II, 1938, p. 329.

[3] Certain of the freezing and thawing test data from series 2 of this investigation were presented in the paper *Effect of Blended Cements and Vinsol Resin-Treated Cements on Durability of Concrete*, by W. F. Kellermann; Journal of the American Concrete Institute, vol. 17, No. 6, June 1946, p. 681.

within the upper 6 percent limit usually specified in current construction practice where air-entraining concrete is used. The results of tests performed on the concrete specimens confirm the conclusions of previous investigations that increased resistance to freezing and thawing is obtained by the use of air-entraining concrete, though at some sacrifice in strength.

It is concluded from the results of the tests that the increased resistance to freezing and thawing of concrete made with blends of portland and natural cements of the types and in the proportions used in this investigation is due to the fact that the concrete contained entrained air.

Many investigations have shown that the upper portion of a plain concrete slab as cast is less resistant to freezing and thawing than the bottom. These tests indicate that air entrainment will tend to improve the uniformity of the concrete throughout the depth of the slab.

GENERAL TEST PROCEDURE

In construction, the general practice is to use a blend consisting of one bag of natural cement and either five or six bags of portland cement. In this investigation all blends were made in the proportion of 14 percent natural cement to 86 percent portland cement, by weight. This is about the proportion of one bag of natural cement weighing 80 pounds to five bags of portland cement weighing 94 pounds each.

The three portland cements used in the investigation were designated by capital letters *A*, *B*, and *C*. All of them were nonair-entraining and as such were designated as *A1*, *B1*, and *C1*. Flake Vinsol resin interground with cements *A* and *B* produced air-entraining portland cements designated as *A2* and *B2*. There was no air-entraining counterpart of cement *C*.

The natural cements used in the investigation were designated by Roman numerals *I* to *V*, inclusive, and the slag cement by Roman numeral *VI*. Two lots of each of the natural cements *II* and *IV*, differing considerably in the amount of interground air-entraining (chloroform-soluble) material they contained, were tested. The samples with the smaller amounts were designated as *IIa* and *IVa*, while the samples with the larger amounts were designated as *IIb* and *IVb*.

In summary, the notation identifying the several portland cements, natural cements, and the slag cement used in the investigation is as follows:

- Portland cements—capital letters *A*, *B*, *C*
  - nonair-entraining—suffix *1*
  - air-entraining—suffix *2*
- Natural cements—Roman numerals *I* to *V*
  - with smaller amount of air-entraining material—suffix *a*
  - with larger amount of air-entraining material—suffix *b*
- Slag cement—Roman numeral *VI*

Results of chemical and physical tests on the cements are given in tables 1 and 2. The results of grading and other physical tests of the aggregates used are shown in table 3.

Table 1.—*Chemical and physical properties of portland cements*

CHEMICAL ANALYSES

	Portland cements				
	A1	*A2*	*B1*	*B2*	*C1*
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Silica (SiO_2)	21.60	21.60	23.90	23.90	21.55
Alumina (Al_2O_3)	6.14	6.29	4.27	4.14	6.02
Ferric oxide (Fe_2O_3)	2.56	2.56	3.28	3.36	3.68
Lime (CaO)	63.90	63.85	64.60	64.65	63.90
Magnesia (MgO)	2.93	2.93	1.09	1.10	1.10
Sulfuric anhydride (SO_3)	1.58	1.61	1.41	1.42	1.58
Sodium and potassium oxide (Na_2O+K_2O)	.68	.54	.64	.61	.68
Loss on ignition	.62	.64	.84	.78	1.42
Chloroform—soluble material [1]	.007	.035	.006	.040	.010

COMPUTED COMPOUND COMPOSITIONS

	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Tricalcium silicate (C_3S)	46	45	44	45	46
Dicalcium silicate (C_2S)	27	28	36	35	27
Tricalcium aluminate (C_3A)	12	12	6	5	10
Tetracalcium alumino—ferrite (C_4AF)	8	8	10	10	11
Calcium sulfate ($CaSO_4$)	2.7	2.7	2.4	2.4	2.7

PHYSICAL PROPERTIES

Apparent specific gravity	3.17	3.21	3.17	3.22	3.19
Specific surface (Wagner) cm.² per gm.	1,730	1,670	1,690	1,735	1,910
Autoclave expansion percent	.15	.17	.01	.02	.05
Normal consisteney percent	23.5	23.0	24.5	24.0	25.0
Time of setting (Gillmore test):					
Initial set hr.-min.	3–10	2–40	3–45	3–30	3–30
Final set hr.-min	4–45	4–10	5–00	5–25	5–00
Tensile strength (1:3 mortar):					
3 days lb. per sq. in.	305	280	285	225	295
7 days lb. per sq. in.	415	370	325	305	360
28 days lb. per sq. in.	480	490	465	385	445
Compressive strength (1:3 mortar):					
3 days lb. per sq. in.	2,015	1,805	1,695	1,420	2,050
7 days lb. per sq. in.	3,645	3,495	2,870	2,395	3,515
28 days lb. per sq. in.	5,450	5,020	4,715	4,045	5,615

[1] A. S. T. M. Method C-114-42 used.

All mixes were designed on the basis of use of nonair-entraining portland cements and no adjustments were made in the proportions when using the air-entraining portland cements or the blends containing natural and slag cements. As a result, the proportions of cement to aggregate by weight were the same for all cement combinations. The mixes were also designed to obtain a constant consistency as measured by the slump test. Observations were made of all of the concrete mixes as to workability. This varied considerably. All

Table 2.—*Chemical and physical properties of natural and slag cements*

CHEMICAL ANALYSES

	Natural cements							Slag cement
	I	*IIa*	*IIb*	*III*	*IVa*	*IVb*	*V*	*VI*
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Silica (SiO_2)	21.15	23.60	23.55	18.15	25.25	25.25	23.75	30.50
Alumina (Al_2O_3)	5.28	7.19	7.40	4.57	5.50	5.50	5.30	9.41
Ferric oxide (Fe_2O_3)	4.32	2.56	2.40	2.08	2.40	2.40	2.80	1.44
Lime (CaO)	33.45	33.05	33.00	46.80	34.45	34.30	31.45	51.15
Magnesia (MgO)	21.65	20.77	20.68	10.45	21.62	21.96	21.65	1.83
Sulfuric anhydride (SO_3)	1.30	1.61	1.96	1.78	2.35	2.25	2.30	.45
Sodium and potassium oxide (Na_2O+K_2O)	1.78	1.78	1.83	1.06	3.00	2.93	3.41	
Loss on ignition	7.90	9.51	9.29	15.07	5.50	5.35	6.40	3.85
Insoluble residue	11.25	13.85	13.85	6.30	11.15	11.05	11.65	.....
Chloroform—soluble material [1]	.05	.04	.32	[2] .33+	.08	.13	18	.....

PHYSICAL PROPERTIES

Apparent specific gravity	3.06	2.90	2.88	2.86	3.03	3.05	3.00	2.80
Specific surface, (Wagner) cm.² per gm.	2,310	2,350	2,445	3,030	2,285	2,035	2,565	2,770
Autoclave expansion percent	[3] 3.8	1.8	2.5	4.2	15.0	15.0	[3] 5.5	—.1
Normal consistency percent	29.0	31.0	32.0	31.0	36.0	38.0	32.0	32.0
Time of setting (Gillmore test):								
Initial set hr.-min.	0–45	3–15	1–10	2–15	1–00	0–55	1–10	2–35
Final set hr.-min.	1–15	5–30	2–35	5–15	1–45	1–45	1–50	5–00
Tensile strength (1:2 mortar):								
3 days lb. per sq. in.	40	35	65	80	[4] 40	[4] 65	75	[4] 60
7 days lb. per sq. in.	50	65	115	125	[4] 90	[4] 90	125	[4] 215
28 days lb. per sq. in.	70	170	235	270	[4] 210	[4] 200	195	[4] 380
Compressive strength (1:2 mortar):								
3 days lb. per sq. in.	135	175	360	510	[4] 185	[4] 215	540	[4] 275
7 days lb. per sq. in.	150	365	655	765	[4] 305	[4] 285	790	[4] 1,060
28 days lb. per sq. in.	430	1,160	1,605	1,790	[4] 855	[4] 710	1,170	[4] 2,190

[1] A. S. T. M. Method C-114-42 used.
[2] All chloroform—soluble material not extracted.
[3] Specimen warped.
[4] 1:3 mortar used.

Table 3.—*Grading and other physical properties of aggregates*

	Potomac River sand used in series 1 and 2	Maryland bank sand used in series 3	Potomac River gravel used in series 1 and 3	Potomac River gravel used in series 2
GRADING				
Percent passing sieve size:				
1½-inch	100	100	100	100
1-inch	100	100	68	100
¾-inch	100	100	43	80
½-inch	100	100	29	50
⅜-inch	100	100	15	30
No. 4	98	94	3	0
No. 8	87	79		
No. 16	76	66		
No. 30	57	44		
No. 50	23	14		
No. 100	5	2		
OTHER PHYSICAL PROPERTIES				
Fineness modulus	2.54	3.01	7.39	6.90
Bulk specific gravity (dry)	2.59	2.57	2.59	2.59
Absorption (24 hour) percent	1.2	1.2	1.1	1.1

of the air-entraining mixtures produced more workable concrete than those that contained only small quantities of air.

The amount of water per unit volume of concrete was calculated from actual yield tests. Differences in water contents reflect differences in the volume of air in the mixes as well as slightly different water requirements of the various cements.

STRENGTH TESTS

The tests comprising series 1 of the investigation were planned to study the flexural and compressive age-strength relation. Mix data for the concretes used in this series are given in table 4. All of the portland cements and blends of the nonair-entraining portland cements with each of the natural and slag cements were used. The mix contained six bags of cement per cubic yard. Potomac River gravel (1½-inch to No. 4) and sand were used as aggregates. Their grading and other physical properties are shown in table 3.

Specimens used in the compression tests were 6- by 12-inch cylinders, and those used in the flexure tests were 6- by 6- by 20-inch beams. The specimens were moist cured and were tested at ages ranging from 3 days to 4 years.

Table 4 shows the calculated air contents for the different concretes. These varied from 1.2 percent for concrete containing nonair-entraining cement *B1* to 5.3 percent for concrete containing blend *A1+IIb*. All of the blends of natural cements and the slag cement produced air contents in excess of 2 percent except natural cement *IIa* and slag cement *VI*. The air-entraining portland cements developed about 3 percent air.

Results of compressive and flexural strength tests for specimens ranging in age from 3 days to 4 years are given in tables 5 and 6, respectively, and are shown graphically in figures 1 to 4, inclusive.

Figures 1 and 2 show the relation between air content and compressive and flexural strength at 28 days. The data at this age were selected for plotting as being typical of the data obtained at all ages. The test data indicate that, for air contents in excess of 2 percent, the compressive strength was reduced approximately in proportion to the increase in the amount of entrained air.

From table 5 it will be noted that for the concrete with the two highest air contents, *A1+IIb* and *A1+IVb*, the compressive strength at 28 days was reduced approximately 30 percent as compared to that obtained with plain portland cement concrete. The corresponding decrease at 4 years was about 24 percent.

In figures 3 and 4 are plotted the age-strength relations for all cement combinations. In each case the data have been plotted in three groups. The left-hand diagrams show the age-strength relations for

Table 5.—*Compressive strength of cylinders,*[1] *series 1 tests*

Cements	Compressive strength[2] at—											
	3 days		7 days		28 days		180 days		360 days		4 years	
	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio
	P. s. i.	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*
A1	2,130	100	4,060	100	5,890	100	6,070	100	7,090	100	7,220	100
A2	1,850	87	3,950	99	5,600	95	6,430	106	6,990	99	7,390	102
A1+I	2,170	102	3,590	90	4,970	84	5,700	94	6,430	91	6,690	93
A1+IIa	2,280	107	3,770	94	5,740	98	6,090	100	7,150	101	7,740	107
A1+IIb	1,690	79	2,840	71	4,290	73	4,670	77	5,490	77	5,510	76
A1+III	2,200	103	3,560	89	5,280	90	6,170	102	6,750	95	7,100	98
A1+IVa	1,830	86	3,190	80	4,750	81	5,490	90	5,980	84	6,550	91
A1+IVb	1,530	72	2,680	67	3,950	67	4,620	76	5,220	74	5,540	77
A1+V	2,070	97	3,330	83	4,970	84	5,580	92	6,250	88	6,630	92
A1+VI	2,000	94	3,720	93	6,200	105			7,570	107		
B1	1,960	100	3,430	100	5,370	100	7,320	100	8,380	100	8,190	100
B2	1,690	86	3,120	91	4,560	85	6,950	95	7,030	84	7,250	89
B1+I	1,900	97	3,270	95	4,850	90	6,500	89	6,980	83	7,910	97
B1+IIa	1,910	97	3,410	99	5,160	96	6,720	92	7,370	88	8,150	100
B1+IIb	1,550	79	2,640	77	3,520	66	5,080	69	5,890	70	6,480	79
B1+III	1,860	95	3,100	90	4,530	84	6,160	84	7,180	86	7,590	93
B1+IVa	1,720	88	2,860	83	4,350	81	5,990	82	6,490	77	7,390	90
B1+IVb	1,330	68	2,360	69	3,610	67	5,110	70	5,630	67	6,310	77
B1+V	1,810	92	2,860	83	4,460	83	6,170	84	6,770	81	7,600	93
B1+VI	1,740	89	3,160	92	5,230	97			7,690	92		
C1	2,410	100	3,860	100	5,880	100	6,870	100	7,510	100	7,570	100
C1+I	2,180	90	3,740	97	5,270	90	6,120	89	7,470	99	7,330	97
C1+IIa	2,230	93	3,540	92	5,260	90	6,570	96	6,760	90	8,150	108
C1+IIb	1,920	80	3,020	78	4,520	77	5,170	75	5,870	78	6,360	84
C1+III	2,120	88	3,600	93	5,240	89	6,300	92	7,130	95	7,790	103
C1+IVa	2,050	85	3,170	82	4,940	84	5,800	84	6,100	81	6,850	90
C1+IVb	1,700	71	2,850	74	4,310	73	5,140	75	5,670	76	6,220	82
C1+V	2,180	90	3,380	88	4,930	84	6,150	90	6,680	89	7,570	100
C1+VI	2,280	95	3,680	95	5,760	98			8,050	107		

[1] Specimens used were 6- by 12-inch cylinders.
[2] The compressive strength values are based on the average of 3 tests in each case. The ratio of strength is based on the strength of plain portland cements *A1*, *B1*, and *C1*, respectively.

Table 4.—*Mix*[1] *data, series 1 tests*

Cements[2]	Water content[3]	W_c[4]	Slump	Flow	Weight of fresh concrete	Actual cement content[3]	Calculated air content
	Gal. per bag		*Inches*	*Percent*	*Lb. per cu. ft.*	*Bags per cu. yd.*	*Percent*
A1	5.2	0.152	2.6	55	149.6	6.0	1.3
A2	5.0	.146	2.8	50	148.1	5.9	2.8
A1+I	5.2	.150	2.9	55	148.0	5.9	2.3
A1+IIa	5.2	.152	2.6	52	149.2	5.9	1.5
A1+IIb	5.1	.144	3.1	58	143.6	5.7	5.3
A1+III	5.2	.149	3.0	54	146.8	5.8	3.0
A1+IVa	5.2	.151	2.8	60	146.8	5.8	3.0
A1+IVb	5.2	.146	2.7	59	143.8	5.7	5.1
A1+V	5.2	.149	2.9	59	146.5	5.8	3.3
A1+VI	5.2	.152	3.0	59	149.3	6.0	1.3
B1	5.1	.150	2.8	52	150.1	6.0	1.2
B2	4.9	.141	3.0	50	148.0	5.9	3.1
B1+I	5.1	.149	2.8	51	148.6	5.9	2.1
B1+IIa	5.1	.150	2.4	46	149.7	6.0	1.3
B1+IIb	5.0	.143	2.8	55	144.8	5.8	4.6
B1+III	5.1	.147	2.7	54	147.4	5.9	2.8
B1+IVa	5.2	.150	2.8	52	147.3	5.9	2.8
B1+IVb	5.2	.147	2.8	57	144.5	5.8	4.6
B1+V	5.1	.147	2.7	53	147.0	5.9	3.1
B1+VI	5.1	.150	2.9	53	149.5	6.0	1.4
C1	5.2	.152	3.0	49	149.4	6.0	1.5
C1+I	5.2	.150	2.9	51	148.1	5.9	2.3
C1+IIa	5.2	.151	2.9	46	149.0	5.9	1.6
C1+IIb	5.1	.144	2.8	48	144.4	5.8	4.8
C1+III	5.2	.149	3.1	51	147.2	5.9	2.8
C1+IVa	5.2	.151	3.1	53	146.8	5.8	3.1
C1+IVb	5.2	.149	2.8	54	144.2	5.7	4.8
C1+V	5.2	.149	2.9	51	146.7	5.8	3.2
C1+VI	5.2	.151	2.7	47	148.9	5.9	1.7

[1] Mix by dry weight, in pounds=94:166:369, using 1½-inch to No. 4 gravel.
[2] Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement, by weight.
[3] Where natural or slag cement was used, 94 pounds was considered as one bag.
[4] Water per unit volume of concrete, based on actual yield tests.

Table 6.—*Flexural strength of beams,[1] series 1 tests*

Cements	Modulus of rupture[2] at—											
	3 days		7 days		28 days		180 days		360 days		4 years	
	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio
	P. s. i.	*Per-cent*	*P. s. i.*	*Per-cent*	*P. s. i.*	*Per-cent*	*P. s. i.*	*Per-cent*	*P. s. i.*	*Per-cent*	*P. s. i.*	*Per-cent*
A1	320	100	460	100	605	100	635	100	660	100	660	100
A2	310	97	470	102	615	102	555	87	600	91	640	97
A1+I	305	95	455	99	535	88	555	87	555	84	605	92
A1+IIa	310	97	430	93	520	86	585	92	620	94	690	105
A1+IIb	275	86	400	87	480	79	490	77	515	78	550	83
A1+III	320	100	430	93	515	85	585	92	625	95	650	98
A1+IVa	320	100	430	93	520	86	505	80	505	77	595	90
A1+IVb	270	84	385	84	460	76	495	78	495	75	570	86
A1+V	305	95	420	91	520	86	540	85	600	91	590	89
A1+VI	330	103	460	100	595	98	.....	.....	670	102	.....	.....
B1	320	100	430	100	560	100	665	100	745	100	735	100
B2	280	88	430	100	550	98	650	98	690	93	625	85
B1+I	305	95	415	97	540	96	625	94	640	86	695	95
B1+IIa	300	94	425	99	545	97	650	98	710	95	710	97
B1+IIb	255	80	390	91	485	87	525	79	640	86	650	88
B1+III	285	89	400	93	550	98	635	95	670	90	725	99
B1+IVa	275	86	370	86	490	87	550	83	660	89	660	90
B1+IVb	225	70	340	79	475	85	510	77	575	77	625	85
B1+V	280	88	385	90	545	97	565	85	635	85	695	95
B1+VI	260	81	425	99	565	101	.....	.....	705	95	.....	.....
C1	380	100	480	100	575	100	640	100	675	100	695	100
C1+I	350	92	480	100	555	97	590	92	640	95	680	98
C1+IIa	340	89	480	100	565	98	580	91	690	102	730	105
C1+IIb	320	84	430	90	510	89	520	81	570	84	625	90
C1+III	350	92	445	93	580	101	605	95	670	99	705	101
C1+IVa	345	91	435	91	575	100	525	82	680	101	650	94
C1+IVb	295	79	380	79	510	89	495	77	550	81	615	88
C1+V	340	89	410	85	520	90	570	89	560	83	680	98
C1+VI	320	84	445	93	585	102	.....	.....	650	96	.....	.....

[1] Specimens used were 6- by 6- by 20-inch beams, tested with third-point loading on an 18-inch span; side as molded in tension (A. S. T. M. designation C-78-44).

[2] The modulus of rupture strength values are based on the average of 3 tests in each case. The ratio of strength is based on the strength of plain portland cements *A1*, *B1*, and *C1*, respectively.

Table 7.—*Mix[1] data, series 2 tests*

Cements[2]	Water content[3]	Slump
	Gal. per bag	*Inches*
A1	6.1	3.3
A2	5.7	3.1
A1+I	6.1	3.8
A1+IIa	6.1	3.4
A1+IIb	5.8	3.9
A1+III	6.0	3.8
A1+V	6.0	4.0
A1+VI	6.0	3.2
B1	5.9	3.5
B2	5.6	3.8
B1+I	6.0	3.3
B1+IIa	6.0	3.1
B1+IIb	5.7	3.1
B1+III	5.8	3.3
B1+V	5.9	3.6
B1+VI	5.9	3.0

[1] Mix by dry weight, in pounds=94:194:339, using 1-inch to No. 4 gravel. Nominal cement factor=5.8 bags per cubic yard.

[2] Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement, by weight.

[3] Where natural or slag cement was used, 94 pounds was considered as 1 bag.

the plain cement concretes, for the air-entraining portland cements (*A2* and *B2* only), and for the slag cement blends. The center diagrams show the corresponding values for concretes containing natural cements *II* and *IV*; that is, the two natural cements that had different amounts of interground air-entraining material. The right-hand diagrams give corresponding data for the other three natural cements.

An inspection of figure 3 reveals that blends of all three plain portland cements with natural cements *IIb* and *IVb* showed considerable reduction in strength compared to the plain portland cement. With the exception of the slag cement *VI*, the other blends gave strengths intermediate between the plain portland and the two high air-entraining natural cements *IIb* and *IVb*. For several ages the blend *A1+IIa* gave higher compressive strengths than the concrete made from plain portland cement.

As will be seen in table 5, the percentage reduction in strength due to air entrainment was less, in all cases, at the 4-year period than at the 28-day period.

The combinations of slag cement *VI* with each of the three portland cements showed approximately as high or higher strengths at 1 year as the portland cements with which they were blended. In general, the air-entraining portland cements showed slightly lower strengths than the companion plain portland cements. The reduction in strength is about what would be expected[4] based on the amount of calculated entrained air.

Blends of low air-entraining natural cements and portland cements gave markedly higher strengths than blends of high air-entraining natural cements and portland cements at all ages of test. This is plainly seen in the center diagram of figure 3.

These general relations hold for all three of the portland cements used. The same trends brought out for compressive strength are characteristic of flexure except that they are not so pronounced: See table 6 and figure 4.

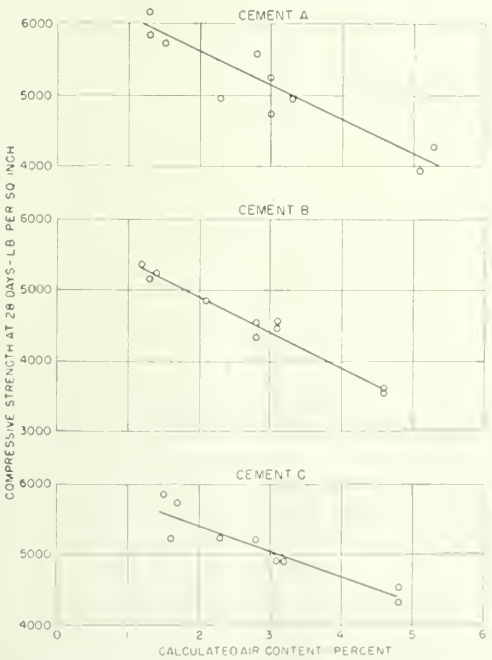


Figure 1.—Relation of compressive strength at 28 days to calculated air content.

[4] *Tests of Concrete Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer*, by H. F. Gonnerman; Journal of the American Concrete Institute, vol. 15, No. 6, June 1944, p. 477.

DRYING SHRINKAGE AND FREEZING AND THAWING TESTS

The tests comprising series 2 of this investigation were designed to study drying shrinkage and resistance to alternate freezing and thawing in water, and to study the effect of initial curing and an interruption in the freezing and thawing cycle. Two of the nonair-entraining portland cements, the two air-entraining portland cements, and blends of the two nonair-entraining portland cements with five of the natural cements and the slag cement were used. The nominal cement factor was 5.8 bags per cubic yard. Mix data for the concretes are given in table 7. Potomac River sand and gravel, of grading and other physical properties shown in table 3, were used as aggregates. The maximum size of the aggregates was limited to 1 inch because of the small size of the test specimens, which were 3- by 4- by 16-inch beams.

For each variable, seven 3- by 4- by 16-inch beams were made for test—three for

Figure 2.—Relation of flexural strength at 28 days to calculated air content.

Table 8.—*Drying shrinkage, series 2 tests*

Cements	Drying shrinkage after storage [1] for—			
	50 days	100 days	150 days	200 days
	Percent	*Percent*	*Percent*	*Percent*
A1	0.045	0.054	0.058	0.060
A2	.048	.058	.061	.062
A1+I	.049	.059	.064	.067
A1+IIa	.051	.062	.066	.068
A1+IIb	.058	.069	.074	.076
A1+III	.049	.059	.064	.065
A1+V	.049	.058	.062	.063
A1+VI	.045	.054	.058	.060
B1	.040	.049	.052	.053
B2	.042	.050	.054	.055
B1+I	.045	.054	.059	.061
B1+IIa	.045	.054	.058	.059
B1+IIb	.046	.055	.059	.061
B1+III	.043	.051	.052	.053
B1+V	.048	.056	.060	.061
B1+VI	.040	.049	.052	.053

[1] Specimens were 3- by 4- by 16-inch beams. They were stored in laboratory air at 70° F., relative humidity 50 percent. Each result is the average of 3 tests.

drying shrinkage, two for freezing and thawing, and two for flexural strength tests as a standard of comparison for the specimens frozen and thawed.

The specimens for drying shrinkage were stored in laboratory air maintained at a constant temperature of 70° F.±2° and constant relative humidity of 50 percent±2 percent. Measurements were made at periodic intervals.

Two groups of freezing and thawing tests were conducted. In the first group the tests were made on specimens cured continuously in moist air for 230 days before freezing was started. Sonic tests were made at periodic intervals on these beams during freezing and thawing,[5] and flexural strength tests were made after 25 cycles of freezing and thawing. Companion beams, moist cured continuously, were also tested for flexural strength at the same age as those subjected to freezing and thawing.

In the second group of tests, two of three specimens previously used for drying shrinkage measurements were, at the conclusion of the drying period, soaked in water for 7 days and then frozen and thawed in water, using the same cycle as was used in the first group. Freezing and thawing was discontinued for 128 days at the end of 60 cycles, after which the cycles of freezing and thawing were again started.

Values for percentage of contraction in air are given in table 8. The initial readings were made after the specimens had been in the mold, under wet burlap, for 24 hours. The specimens were then placed in air storage at 70° F.

[5] Changes in the modulus of elasticity of a concrete beam subjected to alternate freezing and thawing are indicative of the progressive disintegration of the specimen. The modulus of elasticity can be determined by sonic measurement of the specimen's natural frequency of transverse vibration. The specimen is electromechanically vibrated at varying frequencies and an electric pick-up registers the amplitude of vibration, which reaches a well-defined peak at the natural frequency of the specimen. The modulus of elasticity is derived by formula from the weight and dimensions of the specimen and the square of its natural frequency, N^2. Comparison of the effects of freezing and thawing on beams can be made directly on the basis of N^2 when expressed as a percentage change. Details of the sonic test are given in *Application of Sonic Method to Freezing and Thawing Studies of Concrete*, by F. B. Hornibrook; American Society for Testing Materials Bulletin No. 101, December 1939, p. 5.

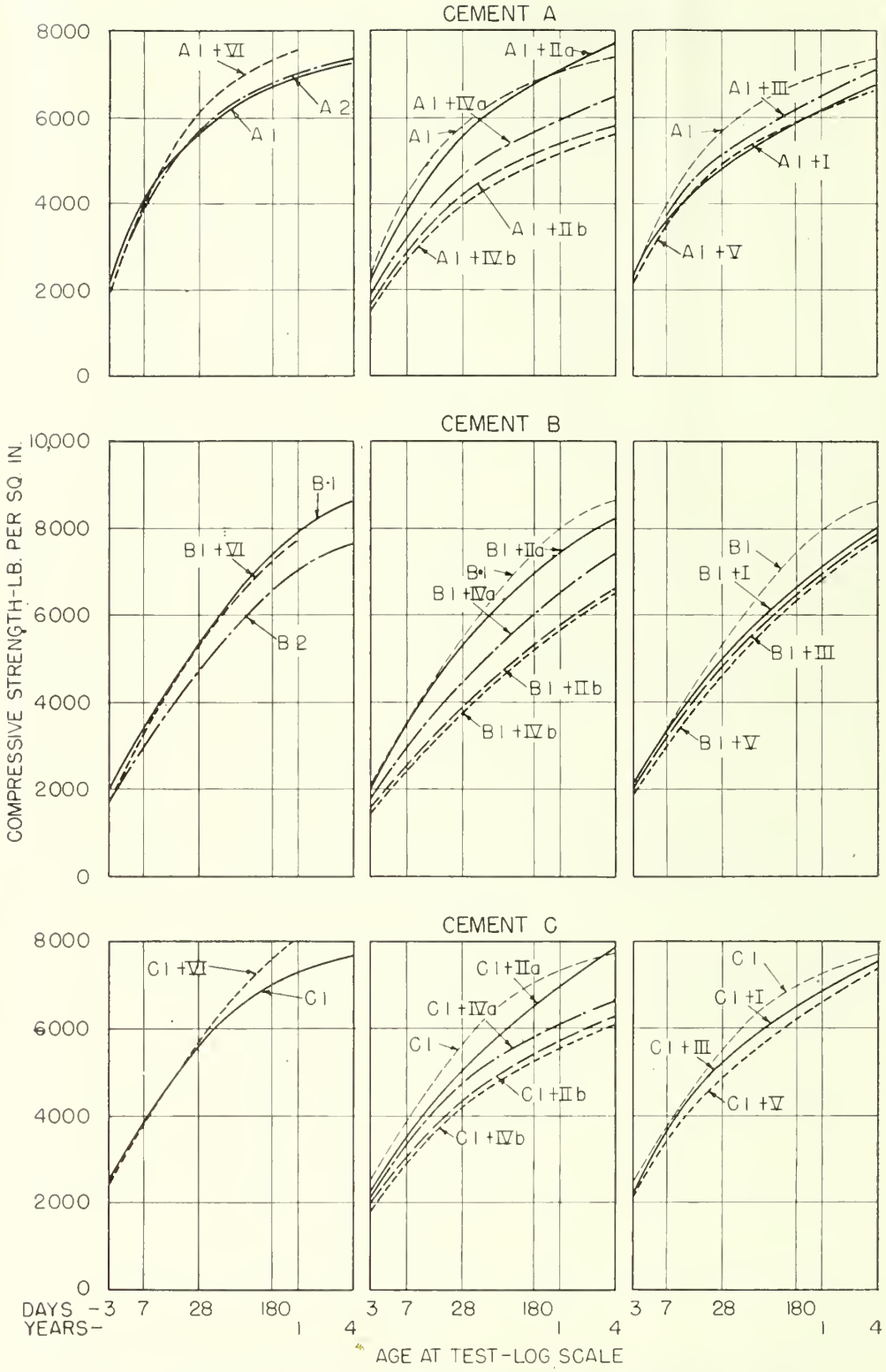


Figure 3.—Effect of blending cements on the compressive strength of concrete.

and 50 percent relative humidity. The values shown in the table were taken from smooth curves.

As may be seen in figure 5, there is no outstanding difference in the shrinkage of concrete made with any of the cements or blends tested. In the case of cement *A1* blended with the air-entrained natural cement *IIb*, the curve indicates slightly greater shrinkage than the other cements tested. It is very doubtful that this small difference is significant. There is also a slight difference in the shrinkage characteristics of concrete containing cement *A* as compared to cement *B*, but this may be a function of their composition. The principal difference in compound composition between the two cements is the tricalcium aluminate content, in which cement *A* is high (12 percent) while cement *B* is low (6 percent).

The first group of freezing and thawing tests was made on 3- by 4- by 16-inch concrete beams cured continuously in moist air for 230 days before freezing started. The specimens were frozen and thawed immersed in water,

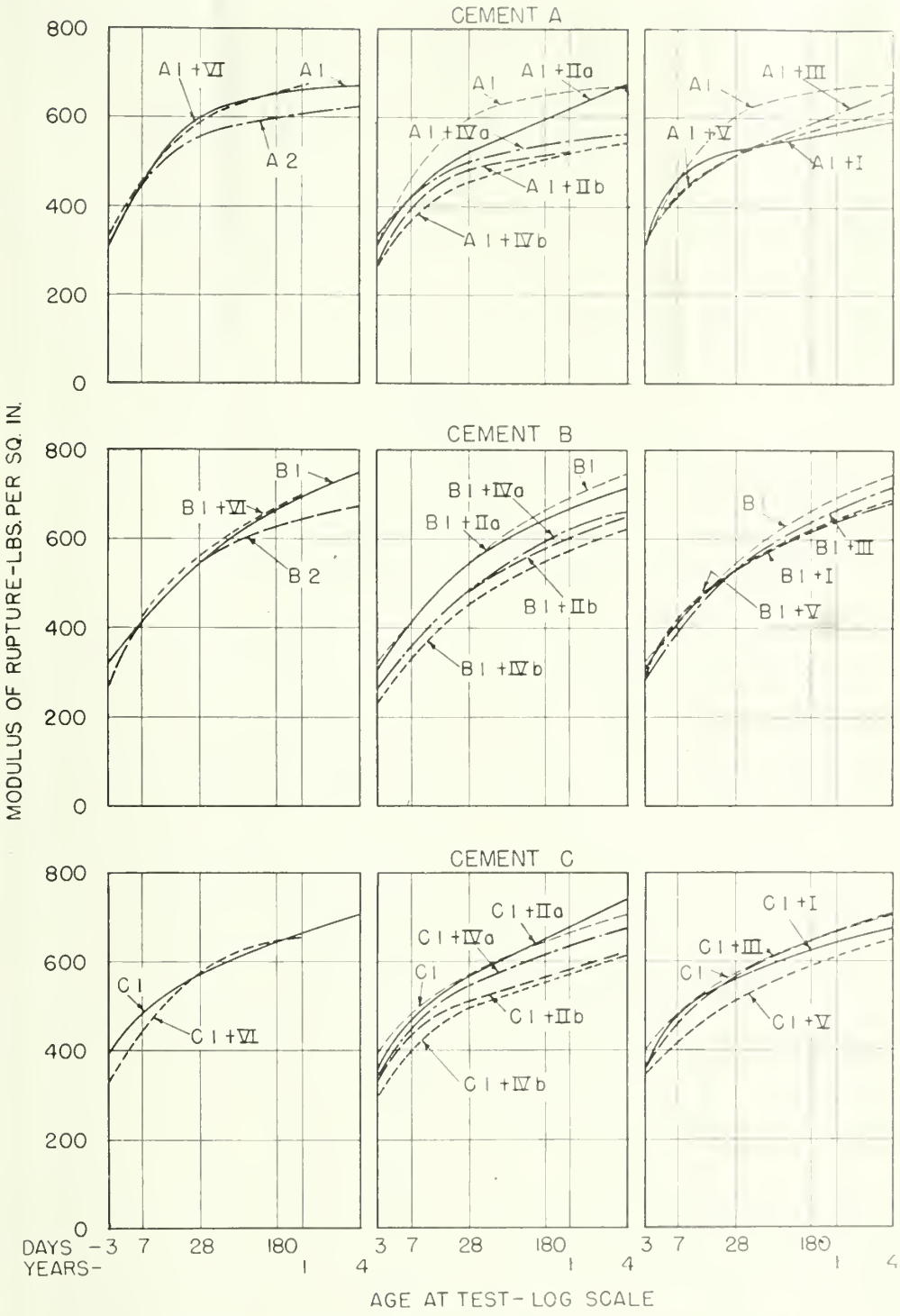


Figure 4.—Effect of blending cements on the flexural strength of concrete.

the temperature range being from 70° to −20° F. The natural frequency of vibration of each specimen was determined sonically at 70° F. just before freezing and thawing started and at the completion of each five cycles of freezing and thawing. The results of these tests are shown in table 9 and are plotted in figure 6. The test was discontinued at the end of 25 cycles because of the marked reduction in natural frequency exhibited by most of the specimens. The specimens were then tested as beams and the flexural strength determined. At the same time continuously moist-cured companion specimens that had not been frozen and thawed were tested, and the reduction in strength caused by freezing and thawing computed.

In table 10 are given the flexural strength data and the percentage drop in sonic modulus of elasticity expressed in terms of the frequency squared, N^2. The values for modulus of rupture of the unfrozen specimens refer to the strength of the specimens that were stored in moist air continuously and tested at the same time the specimens subjected to alternate freezing and thawing were tested. These values are used in computing the values tabulated as reduction in modulus of rupture.

No unit weight determinations were made for the concrete mixes used in the series 2

Table 9.—*Reduction in frequency squared of beams [1] frozen and thawed in water, series 2 tests*

Cements	Reduction in frequency squared [2] expressed in percents after freezing and thawing for—				
	5 cycles	10 cycles	15 cycles	20 cycles	25 cycles
A1	34	43	58	62	72
A2	15	21	24	31	35
A1+I	23	35	37	51	66
A1+IIa	48	69	77	85	88
A1+IIb	19	25	28	41	48
A1+III	13	18	22	29	33
A1+V	15	22	27	31	40
A1+VI	40	61	72	79	82
B1	51	73	84	88	92
B2	12	14	22	25	31
B1+I	32	49	71	74	89
B1+IIa	63	80	88	91	95
B1+IIb	15	23	32	35	44
B1+III	11	17	25	26	33
B1+V	12	19	28	32	38
B1+VI	53	75	85	91	93

[1] The 3- by 4- by 16-inch beams were stored continuously in moist air for 230 days before freezing.
[2] Each value is the average of readings taken on 3- and 4-inch axes of 2 beams.

tests, and therefore it was not possible to compute the air contents of these mixes. In order to obtain data on their relative air contents a supplementary series of tests was run later. These data are shown in table 11. Comparing the calculated values for air content as shown in this table with the corresponding values for the mixes used in series 1 as shown in table 4, it is found that in the cases of those combinations which can be classified as nonair-entraining the values for series 2 are slightly lower than those for series 1. However, the differences average less than 0.5 percent. In the case of the concretes that can be classed as air entraining—those made with cement *A2* and the combinations of *A1* and *B1* with natural cements *IIb*, *III*, and

Table 10.—*Reduction in modulus of rupture and frequency squared of beams [1] after 25 cycles of freezing and thawing in water, series 2 tests*

Cements	Modulus of rupture [2] of—		Reduction in—	
	Unfrozen specimens [3]	Frozen specimens	Modulus of rupture	Frequency squared (N^2)
	Lb. per sq. in.	*Lb. per sq. in.*	*Percent*	*Percent*
A1	790	245	69	72
A2	770	525	32	35
A1+I	730	285	61	66
A1+IIa	705	130	82	88
A1+IIb	695	295	58	48
A1+III	720	430	40	33
A1+V	720	395	45	40
A1+VI	800	180	78	82
B1	780	110	86	92
B2	815	515	37	32
B1+I	870	200	77	81
B1+IIa	780	90	88	91
B1+IIb	745	325	56	44
B1+III	850	445	48	35
B1+V	[4] 850	440	47	39
B1+VI	895	95	89	93

[1] The 3- by 4- by 16-inch beams were stored continuously in moist air for 230 days before freezing.
[2] Beams were tested with center-point loading on a 14-inch span. The beams were tested so that the 3-inch dimension was the depth of the beams as loaded, thus the side of the beam as molded was in tension. Each value is the average of 2 tests unless otherwise noted.
[3] Specimens were stored continuously in moist air at 70° F. until tested, and were tested at the same time freezing and thawing specimens were tested.
[4] Values are based on the average of readings on 3- and 4-inch axes of 2 beams.
[5] One test only.

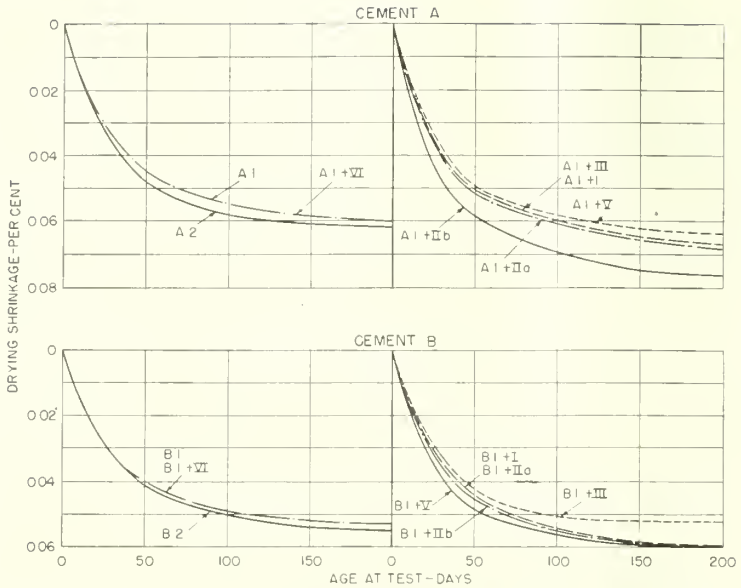


Figure 5.—Effect of blending cements on drying shrinkage of concrete.

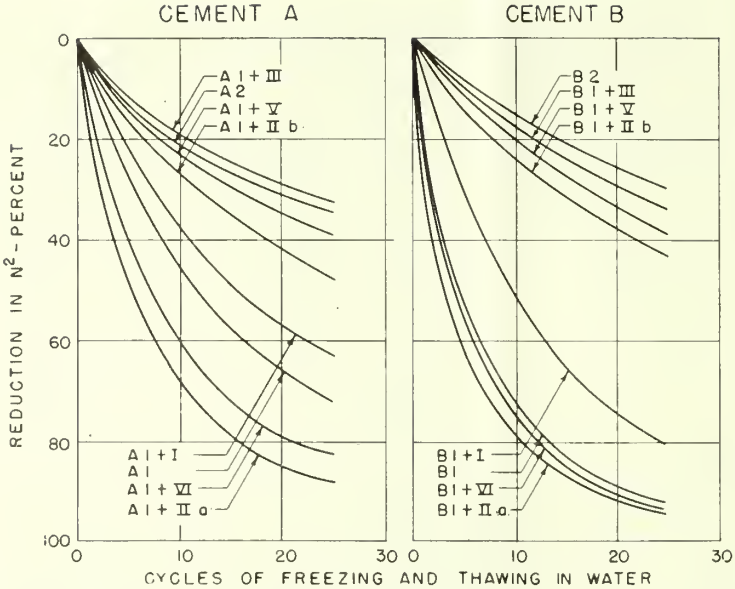


Figure 6.—Effect of blending cements on resistance of concrete to freezing and thawing.

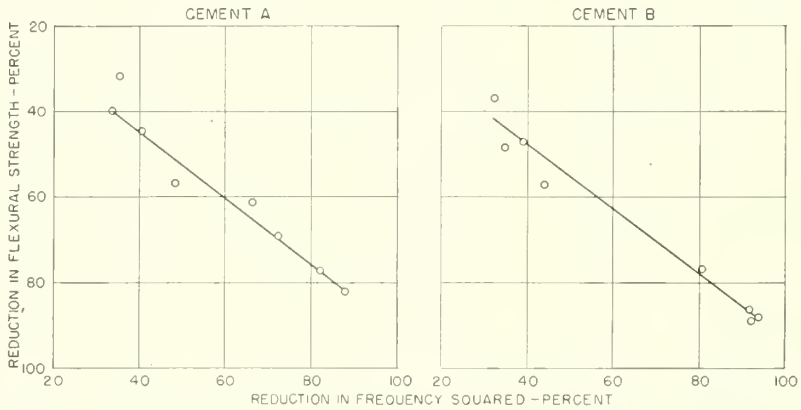


Figure 7.—Relation of reduction in flexural strength to reduction in frequency squared after 25 cycles of freezing and thawing.

V—the values in series 2 are somewhat higher, the average difference being about 1.2 percent. The one exception is air-entraining cement *A2* which shows lower air content in series 2. Cement *B2* was not available for the supplementary test in series 2.

Due to the fact that no determinations of air content were made on the concrete actually used in the freezing and thawing tests, only general reference to the relative air contents as "high" or "low" is made in the discussion of these data. It is felt that the values reported in tables 4 and 11 are sufficiently consistent to warrant this classification.

Referring to the left-hand diagram in figure 6 (which gives the results for cement *A*), it is noted that there is a marked difference in resistance to freezing and thawing resulting from the use of different blends. The greatest reductions in N^2 were found with blends containing natural cement *IIa* and the slag cement *VI*. The plain portland cement *A1* gave results but slightly better. These concretes all had low air contents.

Greatly increased resistance was obtained with the air-entraining portland cement *A2* and with the blends of *A1* with natural cements *IIb*, *III*, and *V*, all of which gave relatively high air contents in concrete. The blends containing natural cement *I* gave intermediate results. The air content of these concretes could be classed as intermediate.

In the right-hand diagram of figure 6 are shown similar curves for cement *B*. The essential difference between these results and those for cement *A* is the poorer resistance shown for the plain portland cement and the blends containing cements *IIa* and *VI*. In this case there is a greater spread between the concretes of poor resistance and those showing good resistance than with cement *A*. The results with blends of cement *B1* confirm the conclusion from the results with cement *A1* that the amount of entrained air is the governing factor between good and poor resistance to the action of freezing and thawing in water. The order of resistance using cement *B1* was, with one exception, the same as that with cement *A1*.

The reduction in natural frequency squared (N^2) was accompanied by a corresponding reduction in flexural strength. This relation is shown in figure 7. The data indicate that the reduction in flexural strength can be predicted with reasonable accuracy from a determination of the reduction in N^2.

Table 11.—*Mix* [1] *data, series 2 supplemental tests* [2]

Cements [3]	Water content [4]	W_c [5]	Slump	Weight of fresh concrete	Actual cement content [4]	Air content	
						Calculated	Measured [6]
	Gal. per bag		*Inches*	*Lb. per cu. ft.*	*Bags per cu. yd.*	*Percent*	*Percent*
A1	5.8	0.169	3.5	147.8	5.8	0.9	1.7
A2	5.7	.168	3.8	146.4	5.8	2.0	2.6
A1+I	5.8	.169	3.6	146.4	5.8	1.7	2.5
A1+IIa	5.8	.169	2.6	147.2	5.8	1.1	1.6
A1+IIb	5.8	.169	4.2	138.9	5.5	6.7	7.3
A1+III	5.8	.169	3.7	142.9	5.7	4.0	5.0
A1+V	5.8	.169	3.5	141.9	5.6	4.7	5.3
A1+VI	5.8	.169	3.2	147.0	5.8	1.2	2.1
B1	5.9	.172	3.6	147.6	5.8	.9	1.4
B2 [7]							
B1+I	5.9	.172	3.1	146.2	5.8	1.7	2.0
B1+IIa	6.0	.174	3.4	147.0	5.8	.9	1.7
B1+IIb	5.9	.172	4.1	139.7	5.5	6.1	6.8
B1+III	5.9	.172	3.5	143.1	5.7	3.7	4.8
B1+V	5.9	.172	3.4	142.3	5.6	4.3	5.5
B1+VI	6.0	.174	3.4	147.0	5.8	.9	1.8

[1] Mix by dry weight, in pounds=94:194:339, using 1-inch to No. 4 gravel.
[2] These mixes were made about 2½ years after the original mixes in series 2 to check air content. The cements were the same as used in the original series.
[3] Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement by weight.
[4] Where natural or slag cement was used, 94 pounds was considered as one bag.
[5] Water per unit volume of concrete, based on actual yield test.
[6] Using Pearson pycnometer.
[7] Cement *B2* was not available.

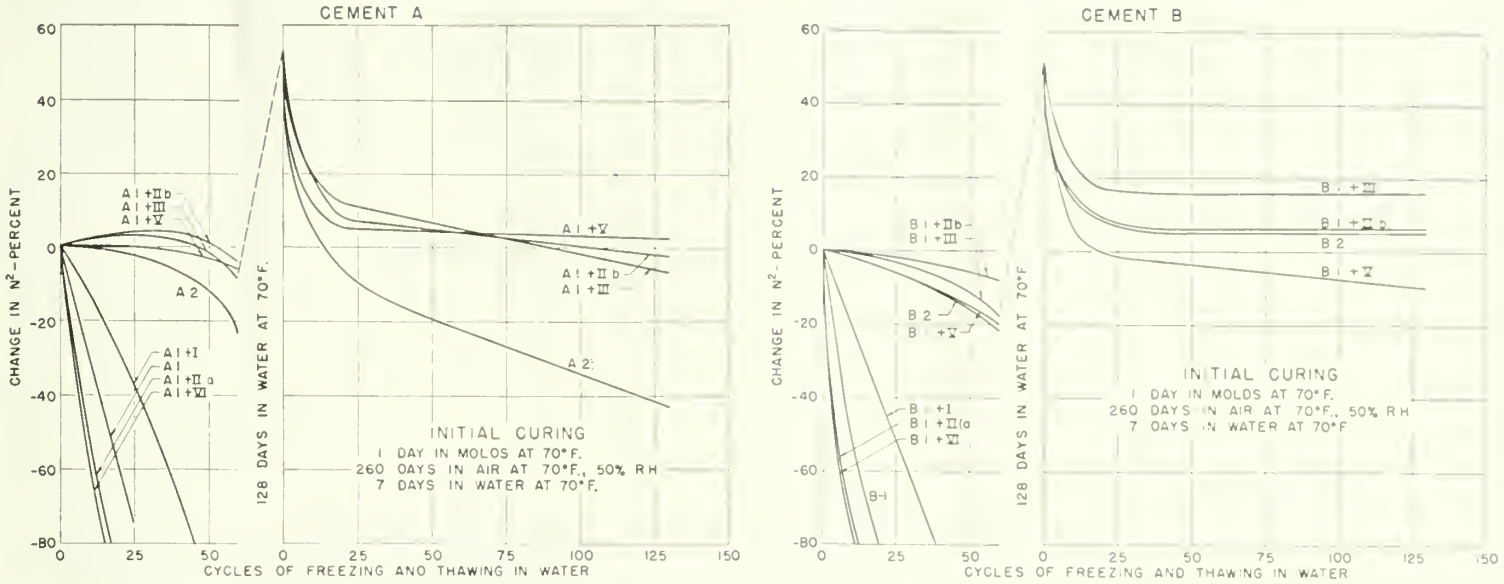


Figure 8.—Effect of blending cements on resistance of concrete to freezing and thawing.

A second set of beam specimens in series 2 (the same specimens used for shrinkage measurements) were cured under wet burlap for 24 hours, followed by storage in laboratory air at 70° F. and 50 percent relative humidity for 260 days. They were then placed in water for 7 days, after which they were frozen and thawed in water, using the procedure previously described. At the end of 60 cycles of freezing and thawing, at which time many of the specimens had disintegrated, the test had to be discontinued temporarily because of other needs for the freezing equipment. Those specimens still remaining were stored in water at 70° F. for 128 days, after which they were again subjected to the freezing and thawing cycle. To simplify discussion, the first group of 60 cycles of freezing and thawing is referred to as the first phase and the second group of freezing and thawing cycles following the 128-day rest period as the second phase.

Results of these interrupted freezing and thawing tests are given in table 12 and are shown graphically in figure 8. The data for cement *A* in table 12 and figure 8 indicate very rapid deterioration during the first phase for the plain portland cement *A1* and the blends which had low air contents. Air-entraining cement *A2* and the blends which had high air contents, however, showed greatly superior resistance in comparison. These combinations showing good resistance were the same as those showing good resistance in the tests made after continuous water curing (see fig. 6). However, the shapes of the curves are reversed. The general trend at 60 cycles was a rapid drop in N^2. As previously explained, those combinations which had not failed at 60 cycles were stored in water for 128 days. At the end of this period they recovered the loss in N^2 indicated at 60 cycles and gained about 45 percent over the initial amount indicated before the first phase of freezing and thawing. As may be seen from the curves for the second phase, the rate in reduction of N^2 was very rapid for the first 20 cycles of freezing and thawing but thereafter, with the exception of the curve for air-entraining cement *A2*, the rate was relatively slow. At 145 cycles of freezing and thawing the value of N^2 for the three blends was about the same as that obtained at the start of the first phase of the test.

The shapes of the curves for the second phase are not the same as those for the first phase but are more like those shown in figure 6. Apparently the resistance to deterioration is influenced by the curing and the moisture condition of the specimens at the time freezing and thawing is started, which accounts for the difference in the shape of the curve. The curves for the second phase of cement *A* in figure 8 and also those shown in figure 6 represent concretes that had been subjected to prolonged moist curing. Conversely, the results for the first phase in figure 8 are for concretes exposed to the drying action of air at 70° F. and 50 percent relative humidity over a long period of time.

It has been shown by other investigations that specimens which have been subjected to cycles of freezing and thawing will, when given a rest period in water, show a recovery in sonic modulus of elasticity.[6]

Weight determinations made during the first phase indicated that all the specimens absorbed water during the first 10 cycles of freezing and thawing and some continued to gain in weight up to 40 cycles. Those specimens that gained the least in weight were those which showed low air contents. They also broke down rapidly in the freezing and thawing tests. Those specimens that gained the most in weight over a longer period were those of relatively high air contents. These gave good resistance in the freezing test. The results indicate that those concretes of low air content were practically saturated after a 7-day immersion period. The concretes having relatively high air contents were not completely saturated at the end of the 7-day im-

[6] *Progress Report, Committee on Durability of Concrete;* Proceedings of the Highway Research Board, vol. 24, 1944, p. 196, fig. 17.

Table 12.—*Effect of interrupted freezing and thawing on reduction of frequency squared of beams [1] frozen and thawed in water, series 2 tests*

Cements	Reduction [2] in frequency squared (expressed in percent) after freezing and thawing													
	Number of cycles in first freezing and thawing phase						Number of cycles in second freezing and thawing phase							
	10	20	30	40	50	60	0	20	40	60	80	100	120	145
A1	28	50	[4]	-	-	-	-	-	-	-	-	-	-	-
A2	2	+6	2	7	15	24	+50	6	17	20	25	36	40	53
A1+I	10	14	44	69	87	[5]	-	-	-	-	-	-	-	-
A1+IIa	46	74	[4]	-	-	-	-	-	-	-	-	-	-	-
A1+IIb	0	+5	+4	+3	+2	4	+44	+8	+8	+1	+4	+4	2	2
A1+III	1	+6	+4	0	1	9	+53	+11	+12	+4	+1	3	6	9
A1+V	2	+1	0	2	3	6	+41	+6	+5	+5	+2	+3	+2	+2
A1+VI	62	97	[6]	-	-	-	-	-	-	-	-	-	-	-
B1	50	78	[4]	-	-	-	-	-	-	-	-	-	-	-
B2	8	3	0	11	16	22	+49	+9	0	+1	+6	+3	+7	+5
B1+I	24	36	70	84	95	[5]	-	-	-	-	-	-	-	-
B1+IIa	75	93	[4]	-	-	-	-	-	-	-	-	-	-	-
B1+IIb	3	+2	3	5	6	7	+40	+10	+12	+7	+10	+7	+8	+6
B1+III	2	+1	3	9	11	18	+51	+17	+15	+15	+22	+17	+17	+18
B1+V	5	2	8	12	17	19	+40	1	1	11	8	12	11	8
B1+VI	80	87	[4]	-	-	-	-	-	-	-	-	-	-	-

[1] Specimens were 3- by 4- by 16-inch beams stored in laboratory air at 70° F. and 50 percent relative humidity for 260 days, followed by 7 days in water prior to start of first freezing.
[2] Each value is the average of readings taken on 3- and 4-inch axes. Plus values indicate an increase in frequency squared.
[3] Specimens were stored in water at 70° F. for 128 days between the first and second freezing and thawing cycle phases.
[4] Failed at 25 cycles. [5] Failed at 50 cycles. [6] Failed at 20 cycles.

Table 13.—*Mix*[1] *data, series 3 tests*

Cements[2]	Water content[3]	Slump
	Gal. per bag	*Inches*
A1	4.8	3.3
A2	4.6	3.7
A1+I	4.8	3.1
A1+IIa	4.8	2.8
A1+IIb	4.7	3.0
A1+III	4.8	2.6
A1+IVa	4.9	2.7
A1+IVb	4.8	2.8
A1+V	4.8	3.3
A1+VI	4.8	3.5
B1	4.8	2.8
B2	4.6	3.3
B1+I	4.8	3.1
B1+IIa	4.8	3.1
B1+IIb	4.7	2.5
B1+III	4.8	2.5
B1+IVa	4.9	3.1
B1+IVb	4.9	2.9
B1+V	4.8	2.8
B1+VI	4.8	3.3

[1] Mix by dry weight, in pounds=94:177:358, using 1½-inch to No. 4 gravel.
[2] Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement, by weight.
[3] Where natural or slag cement was used, 94 pounds was considered as one bag.

Table 14.—*Weight loss of the core tops due to freezing and thawing,*[1] *series 3 tests*

Cements	Weight loss (expressed in percent) after freezing and thawing for—							
	25 cycles	50 cycles	75 cycles	100 cycles	125 cycles	150 cycles	175 cycles	200 cycles
A1	2.4	5.7	8.6	15.5	[2]			
A2	0	.1	.2	.6	1.6	3.1	5.3	7.1
A1+I	.2	.8	1.3	2.8	5.4	7.9	10.7	13.0
A1+IIa	2.1	7.1	[3] 47.2	[4]				
A1+IIb	0	.7	1.9	3.5	6.6	9.2	14.7	19.4
A1+III	0	.2	1.0	2.3	4.4	6.3	9.2	12.3
A1+IVa	.3	.7	1.5	3.1	5.6	7.1	10.1	12.3
A1+IVb	0	0	.5	1.8	4.9	6.9	11.2	15.7
A1+V	0	.2	.8	2.3	4.2	6.1	9.0	11.1
A1+VI	.2	1.3	4.1	[5] 39.9	[3] 49.6	[3]		
B1	1.7	4.3	8.3	11.1	17.4	[3] 48.1	[6]	
B2	.2	.5	1.1	2.0	3.4	5.4	7.4	9.0
B1+I	.6	2.2	4.8	9.4	16.6	28.5	[2] 59.1	[7]
B1+IIa	2.4	5.9	9.8	[3] 69.1	[8]			
B1+IIb	.1	.4	.9	1.9	3.7	6.0	10.3	13.5
B1+III	.6	1.4	1.9	3.1	4.8	6.6	10.2	12.6
B1+IVa	.3	1.3	2.2	3.7	6.6	10.6	15.7	21.0
B1+IVb	0	.1	.6	1.4	2.9	4.5	7.2	9.1
B1+V	0	.2	.8	2.0	3.6	5.4	9.0	11.0
B1+VI	4.2	9.2	26.0	[4]				

[1] Tests were made on 2-inch disks cut from the tops of cores 6 inches in diameter and 8 inches in height. Freezing and thawing was done in a 10-percent calcium chloride solution. Each value is the average of three tests.
[2] Failed at 125 cycles. [3] One specimen disintegrated, and weight loss was assumed to be 100 percent for that specimen.
[4] Failed at 80 cycles. [5] Failed at 130 cycles. [6] Failed at 160 cycles.
[7] Failed at 180 cycles. [8] Failed at 110 cycles.

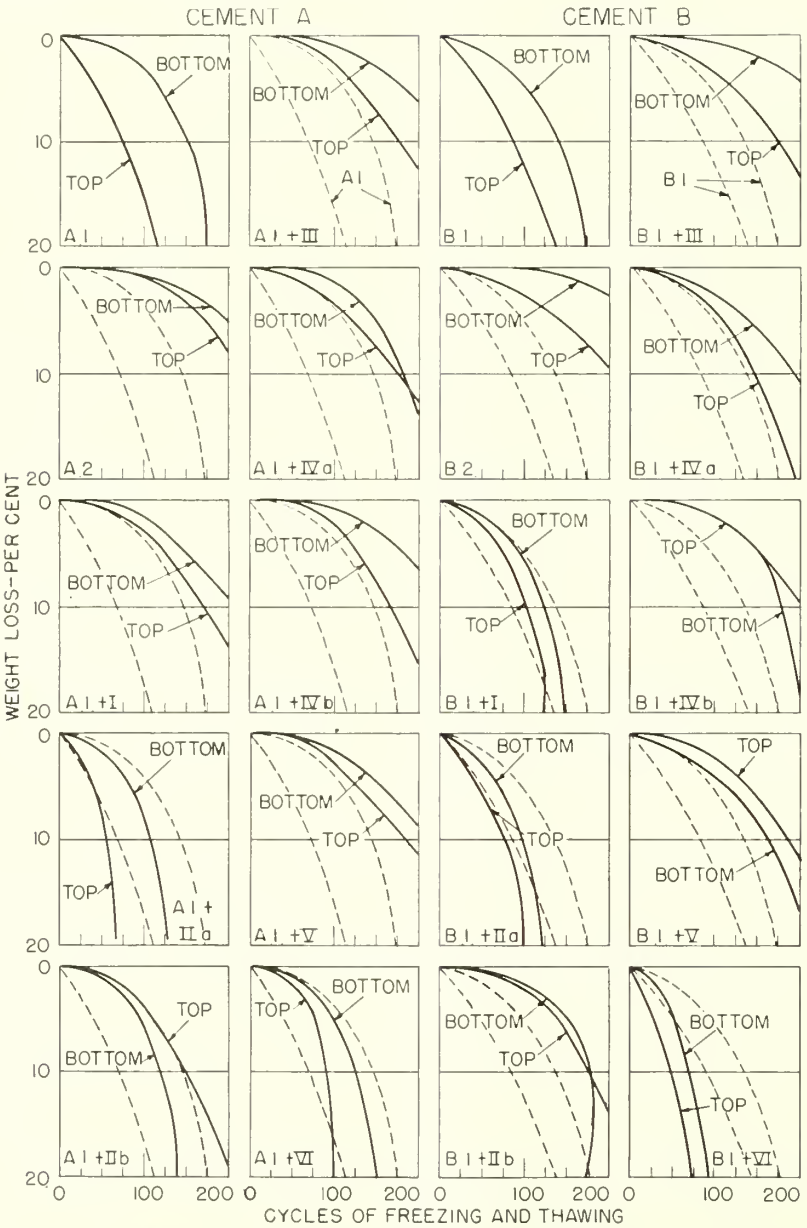


Figure 9.—Weight losses of disks cut from tops and bottoms of frozen and thawed concrete cores.

mersion period and continued to absorb water during the freezing and thawing cycle.

The foregoing observations are offered as an explanation of the shape of the curves for the concretes showing good resistance in the freezing and thawing test during the first phase rather than as an explanation of the difference in behavior between air-entrained and non-air-entrained concrete.

Results shown in figure 8 for blends with cement *B* parallel very closely those shown for cement *A*. The plain cement *B1* and those blends of low air content gave very poor resistance, the order of resistance being exactly the same as with cement *A*. Concretes showing good resistance at the end of 60 cycles of the first phase also exhibited about the same increase in N^2 during the 128-day water storage period as was shown for the corresponding blends of cement *A*.

The data show that interrupting the daily cycle of freezing and thawing and allowing the test specimens to remain unfrozen in water will result in a recovery in sonic modulus.

THE EFFECT OF SEGREGATION

The tests comprising series 3 of the investigation were designed to study the effect of segregation of the constituent materials due to manipulation of the surface in the finishing operations in laboratory-mixed concrete placed to simulate pavement construction practice. Two nonair-entraining and two air-entraining portland cements, and blends of the two former with each of the seven natural cements and the slag cement were used. Mix data are given in table 13. Grading and other physical properties of the aggregates used are shown in table 3. Experience has shown that the typical Potomac River sand grading used in the series 1 and 2 tests does not induce appreciable bleeding or water gain in concrete; and on this account a somewhat coarser sand was used in the series 3 tests. The coarse aggregate was the same as that used in series 1.

The specimens for the tests were slabs 2 feet square and 8 inches thick, cast in forms 2 feet

Table 15.—*Weight loss of the core bottoms due to freezing and thawing,[1] series 3 tests*

Cements	Weight loss (expressed in percent) after freezing and thawing for—							
	25 cycles	50 cycles	75 cycles	100 cycles	125 cycles	150 cycles	175 cycles	200 cycles
A1	0.5	0.9	1.6	2.7	5.7	9.6	53.2	(3)
A2	.4	.5	.6	.7	1.2	1.9	3.5	4.5
A1+I	0	.2	.6	1.5	3.3	5.1	7.2	8.8
A1+IIa	.4	1.1	3.3	7.7	18.5	(4)		
A1+IIb [5]	0	1.5	2.5	4.9	11.1	[2] 38.5	[2] 41.6	[2] 43.7
A1+III	0	0	.2	.8	1.7	2.5	4.8	5.8
A1+IVa	0	.5	.7	1.4	2.8	4.0	9.8	13.6
A1+IVb	.4	.4	.4	.9	1.5	2.3	4.2	5.9
A1+V	.5	.6	.7	1.5	2.3	4.4	7.0	8.4
A1+VI	.5	.8	2.1	6.0	9.7	[2] 41.9	(6)	
B1	.4	2.0	3.5	4.5	7.0	11.6	20.1	[2] 47.1
B2	0	.1	.1	.2	.3	.8	1.7	2.3
B1+I	.2	1.1	2.4	5.8	10.0	15.0	[2] 55.1	[2] 55.7
B1+IIa	.7	2.5	5.3	(7)				
B1+IIb	.4	.4	.5	.9	2.3	3.9	9.2	[2] 13.7
B1+III	0	.2	.4	.6	1.0	1.7	3.1	4.0
B1+IVa	.5	.7	1.4	2.2	3.8	5.4	8.0	10.9
B1+IVb	.1	.5	1.0	1.7	2.9	4.6	11.6	17.7
B1+V	.4	1.8	2.5	3.9	5.7	7.7	12.1	16.7
B1+VI	.9	4.4	12.8	(8)				

[1] Tests were made on 2-inch disks cut from the bottoms of cores 6 inches in diameter and 8 inches in height. Freezing and thawing was done in a 10-percent calcium chloride solution. Each value is the average of three tests except as noted.
[2] One specimen disintegrated, and weight loss was assumed to be 100 percent for that specimen.
[3] Failed at 190 cycles.
[4] Failed at 140 cycles.
[5] Average of 2 tests only.
[6] Failed at 170 cycles.
[7] Failed at 100 cycles.
[8] Failed at 90 cycles.

wide by 4 feet long and screeded with an 8-inch screed weighing 100 pounds per foot.

A tar-paper separator installed full depth midway in the form made it possible to obtain two 2- by 2-foot test slabs without restricting the lateral movement of the concrete caused by the reciprocating motion of the screed. The screed was operated with its long axis parallel to the 4-foot axis of the slabs. Following two passes of the screed, the slabs were belted in a manner similar to that employed in actual road practice. The procedure described was carried out to simulate construction practice and to induce bleeding in the slabs similar to that which frequently occurs in the field. The slabs were cured the first 24 hours under wet burlap in the laboratory, followed by 1 year's curing in the moist room. Five cores 6 inches in diameter were then drilled from each slab. These cores were returned to the moist room for another year of curing, after which 2-inch disks were cut from the tops and bottoms of three of the five cores from each slab. These disks were returned to the moist room for another 15 days' curing, after which they were air-dried at 70° F. and 50 percent relative humidity for 7 days. They were then immersed in a 10-percent calcium chloride solution for 24 hours before being subjected to alternate freezing and thawing in a 10-percent calcium chloride solution. The disks were approximately 750 days old when freezing started.

The weight losses after freezing and thawing are given in table 14 for the disks cut from the tops of the cores, and in table 15 for those cut from the bottoms. The data are shown graphically in figure 9. The dotted-line curves in each chart represent the weight loss curves for the portland cements *A1* and *B1*, and are provided for comparison with weight-loss curves for the blends.

In general, the bottoms of the cores showed more resistance to freezing and thawing in calcium chloride solution than the tops. There were only three exceptions out of 20 cement combinations. The plain portland cement concretes *A1* and *B1* and the blends *A1+IIa*, *A1+IIb*, *A1+VI*, *B1+I*, *B1+IIa*, and *B1+VI* all showed relatively poor resistance to freezing and thawing in calcium chloride solution. Four of the blends, *A1+IIa*, *A1+VI*, *B1+IIa*, and *B1+VI*, had poorer resistance than the comparable plain portland cements, whereas the air-entraining portland cements and the blends of higher air content were of benefit in producing better resistance than was furnished by the plain cement. This confirms the conclusion based on freezing and thawing tests of beam specimens in water. The improvement in resistance over the plain portland cement concrete was generally more pronounced for the tops than for the bottoms of the cores.

The remaining two cores of the original five drilled from each slab were given approximately 15 months' additional moist curing, after which 2-inch disks were cut from the tops and bottoms of each core and tested for specific gravity and absorption. These disks were approximately 830 days old when tested. Upon removal from moist storage they were dried to constant weight at 350° F. They were then boiled for 5 hours and allowed to stand in water for 19 additional hours, after which they were again weighed. From these data, absorptions and specific gravities were calculated by two different methods: First, on the basis of weights resulting from curing for 830 days in moist air and second, on the basis of weights after boiling for 5 hours followed by immersion for 19 hours.

From the results given in table 16 it is readily apparent that the disks from the top showed higher absorption than those from the bottom of the slabs. This was true irrespective of whether plain portland cement, air-entraining portland cement, or blended cement was used. The disks representing air-entraining concrete showed nearly as great a difference between top and bottom as those representing concrete in which the amount of entrained air was low.

The use of air-entraining concrete did affect the ratio of absorption after boiling for 5 hours to absorption due to moist curing for 830 days. Considering the plain portland cements and the blends containing natural cements *IIa*, *IVa*, and *VI*, it will be noted that approximately the same values were obtained for absorption in both tests. Those combinations all had low air contents. The blends containing cements *IIb* and *IVb* gave much higher values for absorption in the 5-hour boiling test than in the 830-day moist curing test. These two blends showed the highest air contents of any concretes tested.

SUMMARY

The following summarizes the test results:

1. Blends containing the nonair-entraining natural cements had higher strengths than blends of air-entraining natural cements at all ages of test.

2. For all combinations the strengths of the concretes were inversely proportional to the air contents.

Table 16.—*Specific gravity and absorption of disks cut from tops and bottoms of cores,[1] series 3 tests*

Cements	Specific gravity,[2] 830 days' moist curing		Absorption,[3] 830 days' moist curing		Specific gravity (dry) [4]		Absorption, 5 hours' boiling	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
			Percent	*Percent*			*Percent*	*Percent*
A1	2.43	2.47	5.06	4.27	2.33	2.37	4.94	4.22
A2	2.36	2.42	5.90	4.56	2.23	2.32	6.39	4.90
A1+I	2.41	2.45	5.38	4.50	2.29	2.34	5.60	4.76
A1+IIa	2.42	2.46	5.51	4.62	2.30	2.35	5.48	4.62
A1+IIb	2.33	2.38	5.85	4.58	2.20	2.27	7.01	5.66
A1+III	2.35	2.41	6.18	4.56	2.22	2.31	6.88	5.26
A1+IVa	2.38	2.43	6.27	4.81	2.21	2.32	6.72	5.17
A1+IVb	2.32	2.37	6.10	4.76	2.18	2.27	7.50	5.64
A1+V	2.36	2.41	5.96	4.83	2.22	2.30	6.80	5.28
A1+VI	2.42	2.46	5.71	4.45	2.29	2.36	5.65	4.70
B1	2.44	2.50	5.56	4.08	2.31	2.40	5.28	4.04
B2	2.31	2.42	5.61	4.34	2.21	2.32	6.74	4.96
B1+I	2.39	2.45	5.96	4.35	2.26	2.35	6.42	4.92
B1+IIa	2.41	2.46	5.98	4.66	2.27	2.35	5.88	4.52
B1+IIb	2.31	2.39	5.76	4.73	2.21	2.29	6.80	5.27
B1+III	2.36	2.44	6.22	4.70	2.22	2.33	6.66	4.80
B1+IVa	2.38	2.44	6.31	4.88	2.24	2.32	6.45	4.92
B1+IVb	2.33	2.41	6.38	4.88	2.19	2.31	7.27	5.10
B1+V	2.35	2.44	6.39	4.89	2.21	2.34	6.98	4.86
B1+VI	2.42	2.48	5.88	4.54	2.29	2.37	5.82	4.38

[1] Specimens were 2-inch disks cut from the tops and bottoms of cores 6 inches in diameter and 8 inches in height. Each value average of two tests.
[2] Bulk specific gravity based on the wet weight after 830 days in moist air.
[3] Based on the wet weight after 830 days in moist air.
[4] Bulk specific gravity based on weight after drying to constant weight at 350° F.
[5] Based on weight after 5-hour boiling and 19 hours in water.

3. The blends of the slag cement with each of the three portland cements had equal or higher strengths at 1 year than the same portland cements without blending.

4. There were no outstanding differences in the drying shrinkage characteristics of concretes made with any of the cement combinations tested.

5. Greatly increased resistance to freezing and thawing in water was obtained with the air-entraining portland cements and with the blends which produced relatively high air contents in concrete.

6. In the interrupted cycles of freezing and thawing in water, only those concretes having relatively high air contents had sufficient resistance to disintegration to survive, as indicated by a drop in N^2 of less than 25 percent in the first phase of 60 cycles. After a further storage period of 128 days in water these same high-air concretes developed further resistance as indicated by a recovery of N^2 that surpassed the initial reading at the start of the first phase. In the subsequent freezing and thawing (second phase) they showed better resistance than they did at 60 cycles in the first phase.

7. Under continued moist curing, concretes with the lower percentages of entrained air absorbed their maximum capacity for water; those containing high percentages of air did not, but the voids filled with water after the drying and boiling treatment.

8. In practically all cases, the bottoms of the cores gave better resistance to freezing and thawing in calcium chloride solution than the tops. The air-entraining portland cements and in general the natural cement blends gave better resistance than the plain portland cements.

New Publications

The Public Roads Administration has recently issued a new bulletin, HIGHWAY STATISTICS, SUMMARY TO 1945, in which is presented historical information of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, highway finance, and mileage. Most of the statistics are carried back over periods of from 20 to 50 years, as available data permitted. A brief explanatory test is included.

The new bulletin provides background for the annual series begun with the issuance of HIGHWAY STATISTICS, 1945. The second annual publication, HIGHWAY STATISTICS, 1946, will be available about December 21. All of these bulletins are for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at the following prices:

HIGHWAY STATISTICS, SUMMARY TO 1945, 40 cents.

HIGHWAY STATISTICS, 1945, 35 cents.

HIGHWAY STATISTICS, 1946, about 35 cents (available after December 21).

WORK OF THE PUBLIC ROADS ADMINISTRATION, 1947, the annual report of the Public Roads Administration for the fiscal year ended June 30, 1947, is now available from the Superintendent of Documents at 20 cents a copy.

U. S. GOVERNMENT PRINTING OFFICE: 1947

Traffic Trends on Rural Roads in 1946

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by THOMAS B. DIMMICK, Highway Economist and MARY E. KIPP, Statistician

Total rural-road traffic in 1946 broke all previous records, exceeding the previous high in 1941 by a slight margin. The seasonal summer peak in 1946, however, was somewhat less than that in 1941.

On the main rural roads of the country, travel in 1946 was over 124 billion vehicle-miles, of which about 80 percent was by passenger cars. Commercial vehicles carried 21 percent more ton-mileage of freight in 1946 than in the previous year, but the proportion of trucks loaded dropped 6 percent. The average weight of carried load remained about the same.

The sharp rise in use of heavier commercial vehicles that occurred during the war was undiminished in 1946. Truck-combination travel was 27 percent higher than in 1941. Axle loads were 31 percent heavier than in the prewar period, and the frequency of heavy axle loads was five times greater. Gross loads have also increased greatly, and heavy loads are far more common than in the prewar period.

TRENDS in total rural traffic reported in this analysis are derived largely from the records received from approximately 670 automatic traffic-recorder stations operated continuously throughout the year on rural roads in 48 States. The automatic-counter records provide no classifications by vehicle type. Such information, and trends in volumes, weights, and characteristics of truck traffic reported here were obtained from the summer survey described in another part of this article. Supplemental counts made by many States yielded valuable information concerning total traffic and the classification of vehicles. Consideration has been given to all available data, but the most reliable of the information, or that derived from the traffic sample with the most complete coverage, is given preference here. In instances where the States have prepared and submitted vehicle-mile estimates for State systems, these have been employed rather than estimates made by applying trend factors to data of previous years.

In figure 1 is demonstrated the variation in rural traffic by months in the year 1941, the peak prewar year; the year 1943, the war year during which traffic was lowest; the year 1945, when traffic increased sharply after VJ-day as gasoline rationing was terminated; and in the year 1946, the first full postwar year. It is interesting to note the differences in the traffic patterns during these years when such varying circumstances existed. While 1946 traffic exceeded that of 1941 by a small percentage, the peak of 1946 traffic was appreciably below that of 1941, when traffic during the summer months was considerably higher than in any other year.

The monthly relations of traffic in the years 1942 through 1946 on all rural roads to that in corresponding months in 1941 are shown in figure 2. Apparent from this chart is the great postwar increase in traffic in the Western States—probably due to the migration of population to the west coast, which was pronounced during the war years. For the same reason, traffic decreased less during the war in the western regions than in other parts of the country. In contrast, traffic in 1946 in the eastern portions of the country generally failed to meet the levels attained in 1941.

Traffic information is analyzed in this report for the United States as a whole, for each census region, and generally for three groups of regions that roughly represent the eastern seaboard, the Central States, and the Western States. The States comprising each of the census regions are indicated in the first column of table 4, page 44. The data available are not deemed sufficient to justify presentation of separate statistics for each State.

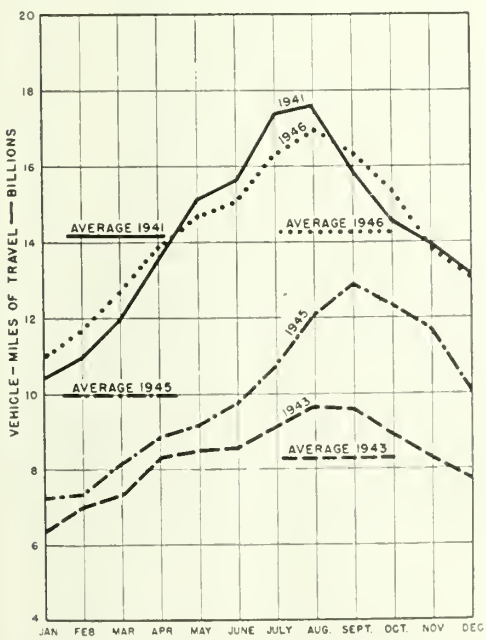


Figure 1.—Vehicle-miles of travel on all rural roads in 1941, 1943, 1945, and 1946, by months.

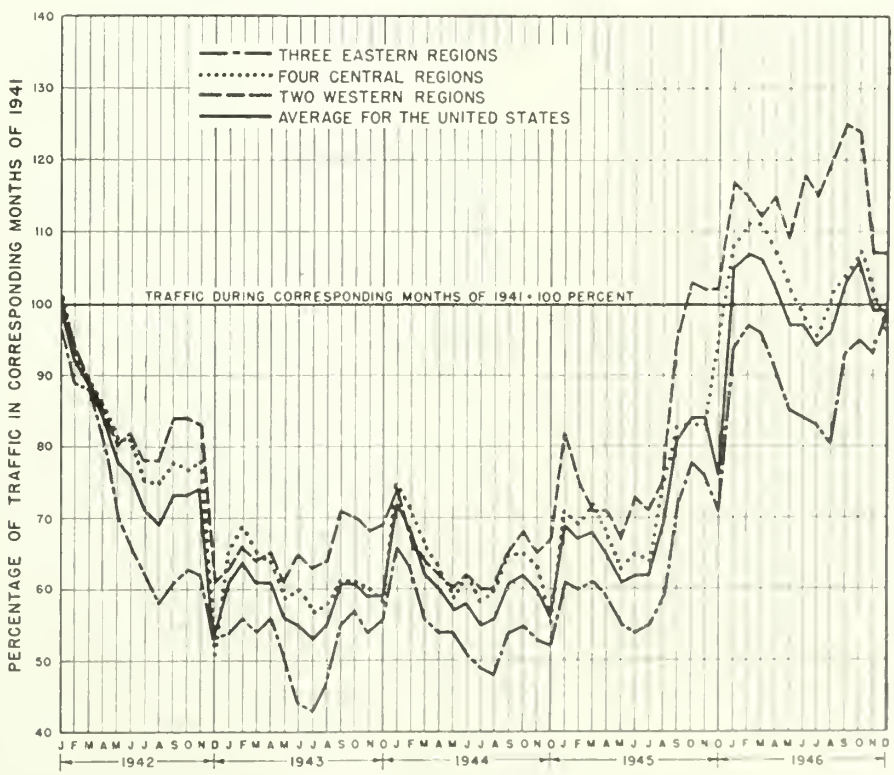

THREE EASTERN REGIONS
FOUR CENTRAL REGIONS
TWO WESTERN REGIONS
AVERAGE FOR THE UNITED STATES
TRAFFIC DURING CORRESPONDING MONTHS OF 1941 • 100 PERCENT
PERCENTAGE OF TRAFFIC IN CORRESPONDING MONTHS OF 1941
140
130
120
110
100
90
80
70
60
50
40
J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D
1942
1943
1944
1945
1946

Figure 2.—Relation of rural traffic in the years 1942–46 to that in corresponding months of 1941, expressed as percentages.

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Table 1.—*Ratio of traffic volume in 1946 to that in corresponding months of 1945, 1943, and 1941, as determined from automatic traffic-recorder data*

RATIO OF 1946 TRAFFIC TO 1945 TRAFFIC

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Eastern regions:												
New England	1.49	1.48	1.55	1.54	1.62	1.65	1.63	1.42	1.26	1.26	1.22	1.29
Middle Atlantic	1.62	1.62	1.56	1.55	1.54	1.56	1.47	1.26	1.32	1.22	1.23	1.57
South Atlantic	1.55	1.63	1.59	1.56	1.55	1.56	1.51	1.42	1.28	1.21	1.22	1.30
Average	1.56	1.61	1.58	1.55	1.56	1.57	1.51	1.35	1.29	1.22	1.23	1.40
Central Regions:												
East North Central	1.64	1.69	1.58	1.70	1.77	1.57	1.52	1.35	1.22	1.31	1.15	1.26
East South Central	1.47	1.61	1.49	1.47	1.51	1.40	1.39	1.31	1.29	1.26	1.23	1.49
West North Central	1.29	1.45	1.42	1.50	1.50	1.49	1.54	1.42	1.23	1.23	1.27	1.40
West South Central	1.58	1.70	1.59	1.52	1.60	1.50	1.49	1.41	1.32	1.33	1.23	1.31
Average	1.51	1.62	1.53	1.57	1.63	1.51	1.50	1.37	1.25	1.28	1.21	1.34
Western regions:												
Mountain	1.38	1.50	1.44	1.52	1.50	1.60	1.57	1.48	1.27	1.14	1.13	1.22
Pacific	1.47	1.54	1.64	1.67	1.70	1.63	1.63	1.66	1.34	1.25	1.02	0.99
Average	1.44	1.53	1.57	1.62	1.63	1.62	1.61	1.60	1.32	1.21	1.05	1.05
United States average	1.52	1.60	1.55	1.58	1.61	1.55	1.52	1.40	1.28	1.25	1.19	1.30

RATIO OF 1946 TRAFFIC TO 1943 TRAFFIC

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Eastern regions:												
New England	1.65	1.60	1.66	1.55	1.67	2.05	2.09	1.77	1.79	1.80	1.84	1.74
Middle Atlantic	1.69	1.70	1.75	1.59	1.73	1.97	1.97	1.64	1.74	1.69	1.74	1.92
South Atlantic	1.82	1.78	1.80	1.69	1.70	1.86	1.86	1.71	1.61	1.64	1.68	1.68
Average	1.74	1.73	1.76	1.63	1.71	1.93	1.94	1.69	1.70	1.69	1.72	1.78
Central regions:												
East North Central	1.61	1.56	1.67	1.67	1.70	1.57	1.60	1.63	1.52	1.68	1.52	1.51
East South Central	1.52	1.55	1.61	1.53	1.52	1.46	1.51	1.73	1.68	1.67	1.66	1.69
West North Central	1.41	1.39	1.50	1.49	1.51	1.57	1.63	1.66	1.62	1.63	1.64	1.55
West South Central	1.72	1.68	1.74	1.67	1.76	1.66	1.69	1.74	1.72	1.77	1.72	1.71
Average	1.58	1.55	1.64	1.61	1.64	1.58	1.61	1.67	1.61	1.69	1.62	1.60
Western regions:												
Mountain	1.61	1.62	1.61	1.58	1.66	1.71	1.80	1.86	1.65	1.64	1.53	1.64
Pacific	2.03	1.87	1.91	1.93	1.92	1.92	1.90	1.94	1.85	1.91	1.66	1.58
Average	1.88	1.78	1.80	1.80	1.83	1.84	1.86	1.92	1.78	1.82	1.61	1.60
United States average	1.67	1.64	1.70	1.64	1.69	1.72	1.75	1.72	1.67	1.71	1.65	1.65

RATIO OF 1946 TRAFFIC TO 1941 TRAFFIC

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Eastern regions:												
New England	0.91	0.83	0.98	0.84	0.77	0.76	0.74	0.70	0.87	0.87	0.78	0.80
Middle Atlantic	.92	.96	.84	.87	.79	.78	.81	.75	.93	.93	.91	1.06
South Atlantic	.97	1.01	1.05	.96	.96	.95	.90	.90	.96	1.02	1.01	1.01
Average	.94	.97	.96	.91	.85	.84	.83	.80	.93	.95	.93	.99
Central regions:												
East North Central	1.07	1.13	1.12	1.05	.98	.92	.87	.96	.99	1.02	.90	.87
East South Central	1.07	1.13	1.13	1.06	1.02	.97	.94	1.09	1.07	1.09	1.12	1.05
West North Central	.99	.98	.99	.98	.91	.95	.94	.94	1.01	1.03	.98	.94
West South Central	1.17	1.22	1.20	1.20	1.21	1.15	1.17	1.16	1.15	1.20	1.16	1.08
Average	1.08	1.11	1.11	1.07	1.02	.98	.95	1.01	1.04	1.07	1.01	.96
Western regions:												
Mountain	1.11	1.12	.99	1.08	1.00	1.06	1.01	1.08	1.11	1.08	.99	1.09
Pacific	1.21	1.17	1.19	1.19	1.14	1.25	1.23	1.27	1.32	1.33	1.11	1.07
Average	1.17	1.15	1.12	1.15	1.09	1.18	1.15	1.20	1.25	1.24	1.07	1.07
United States average	1.05	1.07	1.06	1.02	.97	.97	.94	.96	1.03	1.06	.99	.99

The ratios of traffic volume in 1946 to those in corresponding months of 1945, 1943, and 1941 are shown in table 1. The sharp increase in 1946 traffic over that in the previous year is evident. Because of the end of gasoline rationing in August 1945 and the consequent sharp rise in traffic, the comparative increase in traffic for the later months of 1946 over 1945 is less marked than in the earlier months of the year, particularly in the Western States.

Traffic in 1946 in the western regions, particularly the Pacific States, has consistently exceeded the 1941 levels by a substantial degree. The eastern portion of the country, especially the New England States, has remained generally below the 1941 figure.

The striking rise of 1946 traffic over that of wartime (1943) is evident in all sections of the country.

Table 2 shows the percentage of total monthly vehicle-mileage for each month of the year and for the entire year in each census region in 1941, 1943, and 1946. Notable in this table are the substantial increases in the proportion of the Nation's traffic occurring in the two western regions and, in particular, the Pacific States, during the years 1943 and 1946 from the prewar year 1941. The percentage of the eastern area generally has declined, with the New England States showing the greatest relative decrease, followed by that of the Middle Atlantic States. The central section of the country, on the whole, has remained comparatively stable in this respect; however, the percentages in the south-central portion have increased, while those of the north-central region have decreased.

Table 3 indicates the differences among the annual traffic patterns for the years 1941, 1943, and 1946, showing the percentage of traffic in each month of these years by regions. Normally, for the United States as a whole, the summer months of July and August have higher percentages of traffic than any other month of the year. This is generally true for the years shown; however, summer travel in both 1943 and 1946 was lower in proportion to total yearly traffic than it was in 1941, indicating that prewar summer travel peaks were not yet recovered in 1946.

SUMMER SURVEY ON MAIN RURAL ROADS

During the summer of 1946 the highway departments of 47 States, in cooperation with the Public Roads Administration, conducted a survey to obtain data concerning the volume and composition of traffic and the weights of trucks and truck combinations on rural roads for the determination of trends in traffic. This survey was similar in character to surveys conducted by the several States in 1942, 1943, 1944, and 1945.[1]

The majority of weighing stations were operated during July, August, or September, with Mississippi extending operations through October. This survey was accomplished in every State except South Carolina.

Data were obtained from 541 weighing stations, all of which had been used in the 1945 survey or in the most recent survey made by the State. The work was so scheduled as to insure maximum comparability with information obtained in previous years.

These stations, which were selected initially to effect a representative cross section of traffic on main rural roads, were generally operated for 8 hours on a weekday, either from 6 a. m. to 2 p. m. or from 2 p. m. to 10 p. m. All traffic passing through the stations during the period was counted and classified. Military vehicles, negligible in proportion to total traffic, were not separated from civilian traffic in all States as was done in similar surveys made during the war. Vehicles were classified into the following categories: Local passenger cars; foreign (out-of-State) passenger cars; light, medium, and heavy single-unit trucks; tractor-truck and semitrailer combinations; truck and trailer combinations; and busses.

The survey period, number of stations operated, number of vehicles counted, and number weighed are shown for each State in table 4. A total of 868,095 vehicles were counted during the period of the survey at all stations. In addition, those States that counted military vehicles, if any, separately from civilian, reported a total of 1,606 military vehicles, which was only 0.2 percent of the entire count. These are not shown in any of the

[1] See *Traffic trends on rural roads in 1945*, by T. B. Dimmick; PUBLIC ROADS, vol. 24, No. 10; Oct.-Nov.-Dec. 1946; and *Amount and characteristics of trucking on rural roads*, by J. T. Lynch and T. B. Dimmick; PUBLIC ROADS, vol. 23, No. 9; July-Aug.-Sept. 1943.

Table 2.—*Percentage of total monthly traffic on rural roads in each United States census region in 1941, 1943, and 1946*

Period	Eastern regions				Central regions					Western regions			United States total
	New England	Middle Atlantic	South Atlantic	Total	East North Central	East South Central	West North Central	West South Central	Total	Mountain	Pacific	Total	
January: 1941	4.60	13.10	16.45	34.15	18.32	6.71	11.58	13.48	50.09	5.19	10.57	15.76	100.00
1943	4.10	11.71	14.41	30.22	18.47	7.40	13.03	14.56	53.46	5.87	10.15	16.32	100.00
1946	3.99	11.51	15.30	30.80	18.72	6.86	10.91	15.07	51.56	5.49	12.15	17.64	100.00
February: 1941	5.06	12.54	15.88	33.48	18.36	6.59	12.49	12.87	50.31	5.30	10.91	16.21	100.00
1943	4.07	11.04	14.20	29.31	19.21	7.20	13.48	14.16	54.05	5.77	10.87	16.64	100.00
1946	3.94	11.23	15.09	30.26	19.32	6.97	11.36	14.67	52.32	5.53	11.89	17.42	100.00
March: 1941	5.02	13.05	15.17	33.24	19.11	6.41	12.31	12.60	50.43	5.78	10.55	16.33	100.00
1943	4.77	10.28	14.48	29.53	19.64	6.97	13.19	13.17	53.50	5.93	11.04	16.97	100.00
1946	4.62	10.35	15.07	30.04	20.22	6.78	11.51	14.21	52.72	5.41	11.83	17.24	100.00
April: 1941	5.61	14.59	14.86	35.06	19.87	6.19	12.19	11.26	49.51	5.25	10.18	15.43	100.00
1943	4.93	13.10	13.91	31.94	19.03	6.69	13.05	12.86	51.63	5.97	10.46	16.43	100.00
1946	4.61	12.43	14.00	31.04	20.38	6.39	11.70	13.18	51.65	5.53	11.78	17.31	100.00
May: 1941	5.95	15.38	13.42	34.75	20.58	6.06	12.48	10.31	49.46	5.39	10.40	15.79	100.00
1943	4.86	12.47	13.43	30.76	19.64	6.93	13.27	12.24	52.08	5.91	11.25	17.16	100.00
1946	4.75	12.50	13.24	30.49	20.80	6.38	11.71	12.86	51.75	5.56	12.20	17.76	100.00
June: 1941	6.00	15.21	13.70	34.91	20.28	6.08	12.51	10.74	49.61	5.61	9.87	15.48	100.00
1943	4.01	10.92	12.80	27.73	20.14	7.02	13.67	13.10	53.93	6.41	11.93	18.34	100.00
1946	4.74	12.26	13.55	30.55	19.41	6.10	12.31	12.76	50.58	6.13	12.74	18.87	100.00
July: 1941	6.49	15.22	13.77	35.48	20.81	6.09	12.18	9.84	48.92	5.70	9.90	15.60	100.00
1943	4.32	11.84	12.66	28.82	20.04	6.83	13.17	12.53	52.57	6.20	12.41	18.61	100.00
1946	5.10	13.07	13.22	31.39	19.21	6.06	12.10	12.20	49.57	6.14	12.90	19.04	100.00
August: 1941	6.63	15.87	13.47	35.97	20.60	5.86	12.49	9.78	48.73	5.50	9.80	15.30	100.00
1943	4.74	13.21	12.96	30.91	20.34	6.49	12.79	11.56	51.18	5.94	11.97	17.91	100.00
1946	4.83	12.35	12.62	29.80	20.45	6.65	12.17	11.82	51.09	6.17	12.94	19.11	100.00
September: 1941	6.12	15.27	13.37	34.76	20.36	6.29	12.37	10.76	49.78	5.32	10.14	15.46	100.00
1943	4.87	13.44	13.26	31.57	19.86	6.35	12.53	11.46	50.20	6.03	12.20	18.23	100.00
1946	5.18	13.75	12.53	31.46	19.46	6.49	12.02	11.92	49.89	5.70	12.95	18.65	100.00
October: 1941	6.08	14.53	13.64	34.25	19.85	6.43	12.78	11.26	50.32	5.42	10.01	15.43	100.00
1943	4.78	13.11	13.95	31.81	18.48	6.60	13.16	12.16	50.40	5.99	11.77	17.76	100.00
1946	5.00	12.74	13.17	30.91	19.14	6.61	12.42	12.78	50.95	5.52	12.62	18.14	100.00
November: 1941	5.86	14.14	14.30	34.30	19.76	6.20	12.60	11.82	50.38	5.39	9.93	15.32	100.00
1943	4.16	12.51	14.66	31.33	18.53	6.83	12.68	13.06	51.10	6.04	11.53	17.57	100.00
1946	4.61	12.93	14.62	32.16	18.00	7.02	12.50	13.80	51.32	5.39	11.13	16.52	100.00
December: 1941	5.39	12.94	14.89	33.22	19.79	6.72	12.43	13.11	52.05	4.84	9.89	14.73	100.00
1943	4.19	12.13	15.26	31.58	18.01	6.85	12.66	13.71	51.23	5.56	11.63	17.19	100.00
1946	4.36	13.81	15.24	33.41	17.33	7.14	11.82	14.34	50.63	5.29	10.67	15.96	100.00
Total: 1941	5.82	14.48	14.27	34.57	19.93	6.27	12.38	11.31	49.89	5.40	10.14	15.54	100.00
1943	4.50	12.22	13.77	30.49	19.33	6.82	13.04	12.82	52.01	5.98	11.52	17.50	100.00
1946	4.69	12.48	13.85	31.02	19.41	6.59	11.92	13.17	51.09	5.68	12.21	17.89	100.00

Table 3.—*Percentage of annual traffic in each month in 1941, 1943, and 1946*

Period	Eastern regions				Central regions					Western regions			United States total
	New England	Middle Atlantic	South Atlantic	Total	East North Central	East South Central	West North Central	West South Central	Total	Mountain	Pacific	Total	
January: 1941	4.84	5.54	7.06	6.05	5.63	6.56	5.72	7.30	6.15	5.89	6.39	6.21	6.12
1943	5.86	6.16	6.73	6.37	6.14	6.98	6.42	7.30	6.61	6.31	5.83	5.99	6.43
1946	5.44	5.90	7.06	6.35	6.16	6.65	5.85	7.32	6.45	6.18	6.36	6.30	6.39
February: 1941	5.59	5.57	7.16	6.23	5.93	6.76	6.49	7.33	6.49	6.31	6.93	6.71	6.43
1943	6.36	6.36	7.27	6.77	6.91	7.43	7.28	7.78	7.32	6.80	6.65	6.70	7.05
1946	5.76	6.16	7.46	6.68	6.82	7.21	6.53	7.63	7.01	6.66	6.67	6.67	6.85
March: 1941	6.06	6.33	7.46	6.75	6.73	7.17	6.98	7.82	7.10	7.51	7.30	7.37	7.02
1943	7.80	6.20	7.75	7.14	7.48	7.53	7.41	7.87	7.58	7.30	7.06	7.14	7.37
1946	7.31	6.15	8.08	7.19	7.73	7.63	7.16	8.01	7.66	7.06	7.19	7.15	7.42
April: 1941	7.69	8.05	8.32	8.10	7.96	7.89	7.86	7.95	7.93	7.76	8.03	7.93	7.99
1943	9.15	8.96	8.41	8.76	8.23	8.20	8.36	8.38	8.30	8.31	7.59	7.85	8.36
1946	8.01	8.11	8.21	8.15	8.55	7.90	7.99	8.15	8.23	7.93	7.86	7.88	8.15
May: 1941	9.06	9.43	8.34	8.92	9.16	8.58	8.91	8.11	8.80	8.85	9.10	9.01	8.87
1943	9.23	8.72	8.34	8.62	8.69	8.69	8.70	8.16	8.56	8.45	8.35	8.38	8.55
1946	8.68	8.60	8.21	8.44	9.20	8.31	8.13	8.38	8.69	8.40	8.58	8.53	8.58
June: 1941	9.46	9.65	8.82	9.27	9.35	8.91	9.28	8.72	9.14	9.53	8.91	9.15	9.18
1943	7.70	7.73	8.04	7.87	9.02	8.91	9.07	8.85	8.97	9.28	8.96	9.07	8.65
1946	8.92	8.67	8.63	8.69	8.82	8.15	9.11	8.55	8.73	9.52	9.21	9.31	8.82
July: 1941	11.37	10.71	9.84	10.46	10.65	9.90	10.02	8.87	9.99	10.74	9.95	10.23	10.19
1943	8.83	8.91	8.45	8.69	9.53	9.21	9.28	8.99	9.29	9.53	9.91	9.78	9.19
1946	10.39	9.99	9.10	9.65	9.45	8.76	9.69	8.81	9.26	10.31	10.08	10.15	9.51
August: 1941	11.76	11.33	9.76	10.75	10.68	9.67	10.12	8.93	10.09	10.51	9.88	10.17	10.33
1943	10.21	10.49	9.13	9.83	10.21	9.23	9.51	8.75	9.51	9.63	10.09	9.93	9.70
1946	10.21	9.80	9.03	9.52	10.41	9.99	10.11	8.89	9.91	10.74	10.50	10.58	9.96
September: 1941	9.80	9.84	8.74	9.38	9.53	9.36	9.31	8.87	9.30	9.19	9.32	9.28	9.33
1943	10.47	10.65	9.33	10.02	9.95	9.02	9.30	8.66	9.35	9.77	10.26	10.09	9.68
1946	10.58	10.57	8.67	9.72	9.61	9.44	9.67	8.68	9.36	9.63	10.17	10.00	9.59
October: 1941	8.94	8.60	8.19	8.49	8.54	8.78	8.84	8.53	8.64	8.59	8.16	8.51	8.57
1943	9.46	9.58	9.01	9.32	8.53	8.61	9.00	8.46	8.61	8.91	9.12	9.06	8.92
1946	9.62	9.21	8.57	8.99	8.89	9.04	9.40	8.75	8.99	8.77	9.32	9.14	9.02
November: 1941	8.25	8.01	8.22	8.14	8.13	8.11	8.35	8.58	8.28	8.17	8.03	8.08	8.21
1943	7.66	8.48	8.82	8.52	7.95	8.31	8.06	8.11	8.14	8.38	8.30	8.33	8.29
1946	7.96	8.38	8.54	8.39	7.50	8.61	8.48	8.48	8.13	7.67	7.58	7.17	8.09
December: 1941	7.18	6.94	8.09	7.46	7.71	8.31	7.79	8.99	8.09	6.95	7.57	7.35	7.76
1943	7.27	7.76	8.66	8.09	7.28	7.85	7.58	8.36	7.70	7.27	7.88	7.68	7.81
1946	7.12	8.46	8.41	8.23	6.83	8.28	7.58	8.32	7.58	7.13	6.68	6.82	7.65
Total: 1941	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
1943	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
1946	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 4.—*Survey period, number of stations operated, number of vehicles counted, and number weighed in each State in the special weight survey during the summer of 1946*

Region and State	Survey period	Number of stations	Vehicles counted		Trucks and combinations weighed
			All vehicles	Trucks and combinations	
New England:					
Connecticut	Aug. 5–28	10	27,424	3,886	1,671
Maine	July 29–Aug. 9	8	15,675	2,644	1,167
Massachusetts	do	10	26,631	4,747	2,001
New Hampshire	July 29–Aug. 2	5	12,073	1,437	531
Rhode Island	do	5	9,422	1,979	811
Vermont	Aug. 5–9	5	8,053	602	602
Subtotal		43	99,278	15,295	6,783
Middle Atlantic:					
New Jersey	Aug. 5–21	10	54,419	9,855	2,201
New York	Sept. 23–27	20	33,349	7,933	3,167
Pennsylvania	Aug. 5–13	11	31,666	5,511	1,488
Subtotal		41	119,434	23,299	6,856
South Atlantic:					
Delaware	Sept. 9–12	4	11,434	3,022	464
Florida	June 12–July 5	10	10,725	2,071	1,802
Georgia	Aug. 5–23	10	11,066	2,459	1,777
Maryland	July 29–Aug. 16	10	31,221	6,020	1,040
North Carolina	Aug. 6–23	10	17,115	3,603	2,240
South Carolina	No survey				
Virginia	July 31–Aug. 14	10	15,517	3,838	2,149
West Virginia	Aug. 13–Sept. 5	9	9,585	2,385	1,083
Subtotal		63	106,663	23,398	10,555
Eastern regions, subtotal		147	325,375	61,992	24,194
East North Central:					
Illinois	Aug. 13–Sept. 5	44	69,916	8,173	4,176
Indiana	July 31–Aug. 30	20	33,313	7,330	3,059
Michigan	July 25–Aug. 8	10	24,191	3,616	1,616
Ohio	July 23–Aug. 9	10	19,110	3,257	1,015
Wisconsin	July 25–Aug. 9	12	20,890	3,024	2,379
Subtotal		96	167,420	25,400	12,245
East South Central:					
Alabama	Aug. 14–29	10	9,024	2,025	1,413
Kentucky	July 8–19	10	10,366	2,370	865
Mississippi	July 10–Oct. 30	15	16,272	3,094	737
Tennessee	July 23–Aug. 30	10	8,940	2,215	1,709
Subtotal		45	44,602	9,704	4,724
West North Central:					
Iowa	July 22–31	10	10,037	1,780	1,687
Kansas	Aug. 7–20	10	7,958	1,581	867
Minnesota	Aug. 5–16	8	10,605	2,045	1,164
Missouri	July 29–Aug. 16	14	35,137	6,854	5,469
Nebraska	July 18–Aug. 8	11	9,148	2,000	1,927
North Dakota	Aug. 1–28	10	12,265	2,601	1,901
South Dakota	July 29–Aug. 15	12	5,848	1,018	1,002
Subtotal		75	90,998	17,879	14,017
West South Central:					
Arkansas	Aug. 5–16	10	12,809	3,485	1,553
Louisiana	July 29–Aug. 9	10	7,014	1,859	1,377
Oklahoma	July 25–Aug. 7	10	11,601	2,585	2,498
Texas	July 22–Aug. 19	18	23,780	5,180	3,037
Subtotal		48	55,204	13,109	8,465
Central regions, subtotal		264	358,224	66,092	39,451
Mountain:					
Arizona	May 7–June 7	10	7,439	1,336	594
Colorado	July 9–23	10	20,564	3,501	686
Idaho	Aug. 5–21	12	11,198	1,996	1,280
Montana	July 31–Aug. 28	19	13,573	2,606	2,418
Nevada	July 30–Aug. 13	10	5,527	851	772
New Mexico	July 29–Aug. 14	10	8,979	1,888	1,304
Utah	July 29–Aug. 9	10	11,934	2,318	660
Wyoming	Aug. 6–19	10	7,270	789	745
Subtotal		91	86,484	15,285	8,459
Pacific:					
California	May 29–July 3	20	64,487	10,465	3,738
Oregon	Aug. 12–30	9	16,790	2,755	1,102
Washington	Sept. 4–7	10	16,735	2,182	1,679
Subtotal		39	98,012	15,402	6,519
Western regions, subtotal		130	184,496	30,687	14,978
United States total		541	868,095	158,771	78,623

tables. Trucks and truck combinations numbered 158,771, or 18.3 percent of the total counted. Of these 78,623, or 49.5 percent, were stopped and weighed.

Wherever traffic volume permitted, all civilian trucks and combinations were stopped and weighed, but where this procedure was impracticable, a sample was obtained by weighing at random as many as possible of those which passed the station. Passenger cars and busses were counted but not stopped for weighing purposes. The heavier vehicles, or the single-unit trucks weighing 13 tons or more and combinations weighing 17 tons or more, when stopped, were measured as well as weighed. The type of vehicle, whether it was loaded or empty, and the number and spacing of axles were recorded, as well as the load on each wheel on one side of the vehicle.

COMPARISON OF 1946 AND 1945 SUMMER SURVEYS

The ratios of 1946 summer traffic to corresponding counts in 1945 are given by type of vehicle in table 5. As previously stated, military traffic during 1946 was negligible and is excluded in the present comparison. Particularly outstanding in this table is the striking rise in numbers of passenger cars counted in 1946, especially foreign (out-of-State) passenger cars, which is due in part to the resumption of peacetime vacation travel habits, despite the fact that production of new automobiles has not satiated the demand. Considering the 1946–45 ratio of trucks, generally, travel by single-unit trucks increased more than that by combinations except throughout the Western States where the trend is toward the greater use of the heavier type commercial vehicle.

Table 6 shows the percentage distribution of total traffic by vehicle type in 1946 and 1945. The increases in percentage of passenger cars to total vehicles in 1946 compared with 1945 were largely due to the rise in foreign or out-of-State traffic during 1946.

The average weights of loaded and of empty trucks and combinations in the summer of 1946 and in a corresponding period of 1945 and 1943 are shown in table 7. Contrasting 1946 to 1943, this table indicates that the average weight of loaded single-unit trucks increased 4 percent, that the average weight of loaded combinations increased 10 percent, while the average weight of all loaded trucks and combinations increased 8 percent. Similarly, comparing 1946 with 1945, the average weight of loaded single-unit trucks decreased 3 percent while the average weight of loaded combinations increased 3 percent, and a slight drop was evident in the average weight of all loaded trucks and combinations.

Table 7 indicates that empty weight of the average truck in 1946 declined 4 percent below that of the previous year. Comparing 1946 figures with those of 1943, an increase of 2 percent is evident for the average of all empty trucks. These variations correspond roughly to those shown for loaded vehicles.

VEHICLE-MILES AND TON-MILEAGE OF FREIGHT HAULED

A comparison by census regions of the estimated vehicle-miles of travel on all main rural roads in 1941, 1943, 1945, and 1946 is shown in table 8. Vehicle-miles of all vehicles on main rural roads in 1946 rose sharply in all sections of the country with travel throughout the United States 45 percent over 1945. Truck travel showed an over-all rise of 29 percent and increased in all regions except the Pacific States where single-unit truck travel declined to 88 percent of the previous year's figure. With the rise in passenger-car travel in the first full postwar year, the proportion of truck travel to that of all vehicles generally decreased, in contrast to 1945. In relation to

Table 5.—*Ratio of 1946 summer counts to corresponding counts in 1945*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
All civilian vehicles	1.46	1.45	1.45	1.45	1.48	1.31	1.47	1.48	1.45	1.46	1.42	1.44	1.45
Passenger cars:													
All	1.52	1.54	1.49	1.50	1.53	1.29	1.50	1.53	1.49	1.47	1.58	1.55	1.51
Local	1.38	1.43	1.36	1.39	1.40	1.20	1.29	1.57	1.39	1.05	1.56	1.42	1.39
Foreign	1.96	2.06	1.95	1.98	2.18	1.60	2.56	1.34	1.96	2.37	1.72	2.07	1.98
Trucks and combinations:													
All	1.24	1.27	1.38	1.32	1.25	1.41	1.36	1.39	1.34	1.47	.97	1.11	1.29
Single units	1.25	1.29	1.43	1.35	1.32	1.42	1.36	1.42	1.38	1.44	.88	1.08	1.31
Combinations	1.17	1.22	1.27	1.24	1.12	1.38	1.37	1.29	1.24	1.58	1.11	1.18	1.23
Busses	1.11	.97	.98	.99	1.09	1.11	1.14	.85	1.03	.98	1.11	1.07	1.02

Table 6.—*Percentage distribution by vehicle types in the summer of 1946 and in a corresponding period of 1945*

Region	Passenger cars						Trucks and truck combinations						Busses	
	All		Local		Foreign		All		Single units		Combinations		1946	1945
	1946	1945	1946	1945	1946	1945	1946	1945	1946	1945	1946	1945		
Eastern regions:														
New England	81.73	78.30	56.26	59.35	25.47	18.95	16.71	19.65	13.72	15.92	2.99	3.73	1.56	2.05
Middle Atlantic	79.81	76.56	65.61	66.55	14.20	10.01	19.14	21.88	14.41	16.24	4.73	5.64	1.05	1.56
South Atlantic	76.00	74.03	53.69	57.38	22.31	16.65	22.43	23.63	16.35	16.67	6.08	6.96	1.57	2.34
Average	78.31	75.63	58.71	61.25	19.60	14.38	20.32	22.38	15.21	16.40	5.11	5.98	1.37	1.99
Central regions:														
East North Central	83.65	80.50	62.74	66.36	20.91	14.14	15.57	18.44	10.38	11.59	5.19	6.85	.78	1.06
East South Central	76.51	77.67	53.93	59.16	22.58	18.51	21.54	20.02	17.90	16.56	3.64	3.46	1.95	2.31
West North Central	77.49	75.53	55.20	62.78	22.29	12.75	21.65	23.36	16.92	18.29	4.73	5.07	.86	1.11
West South Central	76.10	73.84	63.31	59.65	12.79	14.19	22.91	24.44	17.38	18.10	5.53	6.34	.99	1.72
Average	79.38	77.27	60.00	62.87	19.38	14.40	19.63	21.33	14.65	15.49	4.98	5.84	.99	1.40
Western regions:														
Mountain	76.91	76.48	37.78	52.36	39.13	24.12	22.01	21.90	18.04	18.24	3.97	3.66	1.08	1.62
Pacific	82.03	73.73	69.99	63.76	12.04	9.97	16.91	24.92	9.49	15.39	7.42	9.53	1.06	1.35
Average	80.34	74.62	59.36	60.05	20.98	14.57	18.59	23.94	12.31	16.31	6.28	7.63	1.07	1.44
United States average	79.21	76.26	59.47	61.83	19.74	14.43	19.66	22.14	14.40	15.93	5.26	6.21	1.13	1.60

Table 7.—*Average weights of loaded and of empty trucks and truck combinations in the summers of 1946, 1945, and 1943*

AVERAGE WEIGHTS OF LOADED TRUCKS

Region	All trucks and combinations					Single-unit trucks					Combinations				
	1946	1945	1943	1946: 1945	1946: 1943	1946	1945	1943	1946: 1945	1946: 1943	1946	1945	1943	1946: 1945	1946: 1943
Eastern regions:	*Pounds*	*Pounds*	*Pounds*			*Pounds*	*Pounds*	*Pounds*			*Pounds*	*Pounds*	*Pounds*		
New England	17,702	18,165	17,499	0.97	1.01	12,317	12,486	12,678	0.99	0.97	35,891	36,864	33,542	0.97	1.07
Middle Atlantic	19,912	21,028	19,786	.95	1.01	13,318	14,140	13,370	.94	1.00	38,215	36,954	35,597	1.03	1.07
South Atlantic	17,809	17,843	17,425	1.00	1.02	10,991	11,368	11,122	.97	.99	33,512	33,079	31,402	1.01	1.07
Average	18,609	19,040	18,255	.98	1.02	12,092	12,583	12,121	.96	1.00	35,361	34,781	33,000	1.02	1.07
Central regions:															
East North Central	22,625	24,283	21,588	.93	1.05	13,043	13,511	12,641	.97	1.03	35,776	34,449	33,313	1.04	1.07
East South Central	15,405	15,684	14,383	.98	1.07	10,784	11,556	10,576	.93	1.02	30,729	29,410	25,825	1.04	1.19
West North Central	16,968	16,779	15,734	1.01	1.08	10,282	10,386	10,383	.99	.99	34,023	33,252	29,524	1.02	1.15
West South Central	15,691	14,126	14,467	1.11	1.08	9,558	8,794	8,297	1.09	1.15	29,745	28,250	27,608	1.05	1.08
Average	18,521	18,768	17,301	.99	1.07	11,061	11,104	10,591	1.00	1.04	33,500	32,389	30,497	1.03	1.10
Western regions:															
Mountain	18,453	17,762	16,073	1.04	1.15	10,759	10,460	9,974	1.03	1.08	40,186	39,082	39,647	1.03	1.01
Pacific	24,486	23,161	21,551	1.06	1.14	12,714	13,292	11,388	.96	1.12	49,186	46,821	45,061	1.05	1.09
Average	22,621	21,928	19,293	1.03	1.17	11,930	12,483	10,770	.96	1.11	47,440	45,693	43,345	1.04	1.09
United States average	19,300	19,539	17,918	.99	1.08	11,580	11,890	11,137	.97	1.04	37,373	36,409	33,879	1.03	1.10

AVERAGE WEIGHTS OF EMPTY TRUCKS

Eastern regions:															
New England	8,413	9,324	8,194	0.90	0.99	6,811	7,872	7,016	0.87	0.97	20,320	18,898	17,669	1.08	1.15
Middle Atlantic	9,939	10,197	8,659	.97	1.15	7,786	7,890	6,795	.99	1.15	18,362	18,767	16,675	.98	1.10
South Atlantic	7,236	7,383	8,384	.98	.86	5,498	5,603	5,903	.98	.93	15,753	15,585	15,004	1.01	1.05
Average	8,316	8,746	8,493	.95	.98	6,444	6,823	6,365	.94	1.01	17,093	17,154	15,788	1.00	1.08
Central regions:															
East North Central	9,426	10,042	9,104	.94	1.04	6,623	7,599	6,794	.87	.97	17,927	17,266	16,812	1.04	1.07
East South Central	5,499	7,022	6,501	.78	.85	5,456	5,832	5,493	.94	.99	14,393	15,573	12,384	.92	1.16
West North Central	8,079	7,789	7,404	1.04	1.09	5,837	5,789	5,587	1.01	1.04	17,255	16,907	14,802	1.02	1.17
West South Central	7,195	7,062	6,286	1.02	1.14	5,341	5,201	4,752	1.03	1.12	15,196	14,489	12,956	1.05	1.17
Average	7,173	8,069	7,594	.96	1.02	5,780	6,078	5,740	.95	1.01	16,459	16,054	15,108	1.03	1.09
Western regions:															
Mountain	7,012	6,653	6,544	1.05	1.07	5,322	5,162	4,923	1.03	1.08	19,150	19,919	19,855	.96	96
Pacific	12,022	11,549	9,725	1.04	1.24	8,011	8,128	6,155	.99	1.31	23,843	22,861	19,326	1.04	1.23
Average	9,574	9,599	8,414	1.00	1.14	6,537	6,804	5,616	.96	1.16	22,693	22,314	19,491	1.02	1.16
United States average	8,201	8,558	8,017	.96	1.02	6,104	6,441	5,932	.95	1.03	17,793	17,517	15,864	1.02	1.12

Table 8.—*Comparison of estimated 1946 vehicle-miles of travel on main rural roads with corresponding figures for 1941, 1943, and 1945*

Region and year	All vehicles, vehicle-miles	All trucks and combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and combinations	Vehicle-miles	Percentage of all trucks and combinations	Vehicle-miles
Eastern regions:							
New England:	*Thousands*		*Thousands*		*Thousands*		*Thousands*
1941	6,663,830	14.7	980,695	90.5	887,783	9.5	92,912
1943	2,961,043	21.8	645,154	82.3	531,100	17.7	114,054
1945	3,928,601	19.6	769,497	81.0	623,565	19.0	145,932
1946	5,713,086	16.7	953,902	82.1	783,031	17.9	170,871
1946: 1945	1.45	.85	1.24	1.01	1.26	.94	1.17
Middle Atlantic:							
1941	17,108,107	20.2	3,447,631	81.0	2,793,416	19.0	654,215
1943	8,355,926	24.9	2,078,950	73.9	1,536,083	26.1	542,867
1945	10,725,475	21.8	2,337,486	74.2	1,735,098	25.8	602,388
1946	15,532,600	19.1	2,968,335	75.3	2,234,986	24.7	733,349
1946: 1945	1.45	.88	1.27	1.01	1.29	.96	1.22
South Atlantic:							
1941	18,612,782	23.2	4,315,146	77.8	3,357,800	22.2	957,346
1943	10,365,819	28.4	2,946,837	69.2	2,038,327	30.8	908,510
1945	12,835,292	23.4	3,007,162	70.6	2,121,565	29.4	885,597
1946	18,439,919	22.5	4,149,173	72.9	3,025,426	27.1	1,123,747
1946: 1945	1.44	.96	1.38	1.03	1.43	.92	1.27
Subtotal:							
1941	42,384,719	20.6	8,743,472	80.5	7,038,999	19.5	1,704,473
1943	21,682,788	26.2	5,670,941	72.4	4,105,510	27.6	1,565,431
1945	27,489,368	22.2	6,114,145	73.3	4,480,228	26.7	1,633,917
1946	39,685,605	20.3	8,071,410	74.9	6,043,443	25.1	2,027,967
1946: 1945	1.44	.91	1.32	1.02	1.35	.94	1.24
Central regions:							
East North Central:							
1941	24,192,260	17.8	4,305,309	68.0	2,926,163	32.0	1,379,146
1943	13,681,965	20.8	2,839,057	64.0	1,817,123	36.0	1,021,934
1945	16,018,709	18.4	2,940,681	62.9	1,848,320	37.1	1,092,361
1946	23,563,034	15.6	3,664,837	66.7	2,444,116	33.3	1,220,721
1946: 1945	1.47	.85	1.25	1.06	1.32	.90	1.12
East South Central:							
1941	7,533,991	21.7	1,632,240	90.0	1,469,033	10.0	162,207
1943	4,771,709	21.4	1,023,092	82.0	835,937	18.0	187,155
1945	5,423,111	20.9	1,134,490	82.7	938,635	17.3	195,855
1946	7,470,206	21.5	1,602,875	83.1	1,332,105	16.9	270,770
1946: 1945	1.38	1.03	1.41	1.00	1.42	.98	1.38
West North Central:							
1941	15,260,119	20.8	3,172,121	80.8	2,563,019	19.2	609,102
1943	9,201,549	26.0	2,392,452	78.4	1,876,302	21.6	516,150
1945	10,180,874	23.3	2,367,392	78.3	1,853,832	21.7	513,560
1946	14,971,991	21.5	3,216,942	78.2	2,514,597	21.8	702,345
1946: 1945	1.47	.92	1.36	1.00	1.36	1.00	1.37
West South Central:							
1941	13,745,126	21.8	2,994,305	78.9	2,362,360	21.1	631,945
1943	8,997,058	29.2	2,625,956	74.1	1,946,830	25.9	679,126
1945	10,724,349	24.1	2,583,820	74.1	1,914,077	25.9	669,743
1946	15,721,893	22.8	3,591,054	75.9	2,724,193	24.1	866,861
1946: 1945	1.47	.95	1.39	1.02	1.42	.93	1.29
Subtotal:							
1941	60,731,496	19.9	12,103,975	77.0	9,320,575	23.0	2,783,400
1943	36,652,281	24.2	8,880,557	72.9	6,476,192	27.1	2,404,365
1945	42,347,043	21.3	9,026,383	72.6	6,554,864	27.4	2,471,519
1946	61,727,124	19.6	12,075,708	74.7	9,015,011	25.3	3,060,697
1946: 1945	1.46	.92	1.34	1.03	1.38	.92	1.24
Western regions:							
Mountain:							
1941	6,741,489	19.5	1,316,246	88.0	1,158,270	12.0	157,976
1943	4,315,357	24.8	1,069,674	85.3	912,433	14.7	157,241
1945	5,194,915	21.5	1,118,716	83.3	931,905	16.7	186,811
1946	7,476,867	21.9	1,639,583	82.0	1,343,755	18.0	295,828
1946: 1945	1.44	1.02	1.47	.98	1.44	1.08	1.58
Pacific:							
1941	12,647,141	16.0	2,021,200	76.1	1,538,563	23.9	482,637
1943	8,373,365	20.8	1,738,241	67.8	1,178,086	32.2	560,165
1945	10,761,149	24.6	2,647,932	61.7	1,634,744	38.3	1,013,188
1946	15,259,494	16.8	2,559,106	56.1	1,435,685	43.9	1,123,421
1946: 1945	1.42	.68	.97	.91	.88	1.15	1.11
Subtotal:							
1941	19,388,630	17.2	3,337,446	80.3	2,696,833	19.2	640,613
1943	12,688,722	22.1	2,807,925	74.5	2,090,519	25.5	717,406
1945	15,956,064	23.6	3,766,648	68.1	2,566,649	31.9	1,199,999
1946	22,736,361	18.5	4,198,689	66.2	2,779,440	33.9	1,419,249
1946: 1945	1.42	.78	1.11	.97	1.08	1.06	1.18
United States total:							
1941	122,504,845	19.7	24,184,893	78.8	19,056,407	21.2	5,128,486
1943	71,023,791	24.4	17,359,423	73.0	12,672,221	27.0	4,687,202
1945	85,792,475	22.0	18,907,176	71.9	13,601,741	28.1	5,305,435
1946	124,149,090	19.6	24,345,807	73.3	17,837,894	26.7	6,507,913
1946: 1945	1.45	.89	1.29	1.02	1.31	.95	1.23

1941, the last prewar year, all-vehicle travel on main rural roads showed a gain of 1 percent in 1946, and although single-unit-truck travel in 1946 was only 94 percent of that in 1941, travel of truck combinations was 27 percent above the 1941 figure, evidencing a sharp rise in the use of heavier commercial vehicles. This trend was apparent during the war years when the proportion of truck travel performed by truck combinations remained consistently higher than in prewar years.

Table 9 shows the estimated percentage of trucks and truck combinations loaded the, average carried load, and the ton-miles carried on main rural roads in 1941, 1943, 1945, and 1946. For the country as a whole in 1946, the ton-mileage of all trucks and combinations increased 21 percent above the previous year, the average weight of the carried load remained the same, while the percentage of trucks loaded dropped from 55.1 percent to 51.7 percent. Ton-mileage of both single-unit trucks and truck combinations increased in all census regions in 1946 except the Pacific States, where single-unit ton-mileage was 77 percent of that of 1945. The mountain region showed the greatest rise in ton-mileage in comparison to 1945, it being 40 percent for single-unit trucks and 50 percent for combinations. While the use of heavier vehicles has increased since 1941, as mentioned above, the average carried load has increased 18 percent, and the ton-mileage hauled, 38 percent.

MAXIMUM AXLE AND GROSS LOADS

Table 10 shows the average of the maximum axle loads of the loaded vehicles weighed during the summer survey in 1946 and during those of previous years. Axle loads have remained approximately the same during the current year as in the prior war years; however, in comparison to prewar years, average maximum axle loads in 1946 for both truck classifications shown are substantially heavier. Since the prewar period, the average maximum axle load for the entire United States has increased 31 percent.

Paralleling the increase in the average maximum axle load, the frequency of heavier axle loads is on the whole five times greater than in the prewar period when the original survey was made (1936–40). This is demonstrated in the left half of table 11 which shows the number of axle loads of 18,000, 20,000, and 22,000 pounds or more per 1,000 trucks and combinations counted in summer surveys of 1946, 1945, 1943, and the original prewar survey. In each category shown, the highest frequency of heavy axle loads was found in the Middle Atlantic States. Heavy axle loads of 20,000 pounds and more are generally more frequent in that area and the New England States, where higher maximum axle loads are permitted by law, than in other sections of the country.

The frequency of heavy vehicles is shown in the right half of table 11. Here it is apparent again that heavier vehicles have come into increasingly wider use throughout the war period and are now far more common than in the prewar period. Although in 1946 an over-all slight drop from 1945 is found in gross weights exceeding 30,000 pounds, frequencies in the categories 40,000 pounds and over and 50,000 pounds and over have risen slightly in this same period. In the two heaviest of the three weight classes shown, the Pacific States, having liberal maximum gross weight restrictions, lead all other regions.

FREQUENCY OF HEAVY LOAD CONCENTRATIONS

In order that the effect of heavy axle loads and heavy total loads may be considered in relation to longitudinal distribution, loads and axle spacings have often been combined by the use of a so-called gross load formula, $W=C(L+40)$, in which W is the total weight of the vehicle in pounds, or the weight of an interior group of axles, and L is the distance in feet between the first and last axle of the vehicle, or of any interior group of

Table 9.—*Comparison of the estimated percentage of trucks and truck combinations loaded, average carried load, and ton-miles carried on main rural roads, with corresponding figures for 1941, 1943, and 1945*

Region and year	All trucks and combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Carried load		Percentage loaded	Carried load		Percentage loaded	Carried load	
		Average weight	Ton-miles		Average weight	Ton-miles		Average weight	Ton-miles
		Tons	*Thousands*		*Tons*	*Thousands*		*Tons*	*Thousands*
Eastern regions:									
New England:									
1941	72.9	2.79	1,993,120	72.7	2.22	1,435,923	74.3	8.06	557,197
1943	56.3	3.89	1,378,333	52.8	2.34	655,257	72.5	8.75	723,076
1945	53.8	3.92	1,623,664	50.9	2.27	721,397	66.4	9.31	902,267
1946	52.4	3.99	1,995,842	48.2	2.24	844,357	57.2	9.40	1,151,485
1946 : 1945	.97	1.02	1.23	.95	.99	1.17	.86	1.01	1.28
Middle Atlantic:									
1941	70.3	3.41	8,267,429	69.9	2.14	4,175,576	72.1	8.67	4,091,853
1943	57.4	4.16	4,963,933	54.5	2.24	1,879,681	65.4	8.69	3,084,252
1945	54.6	4.30	5,487,126	51.9	2.36	2,129,030	62.3	8.95	3,358,096
1946	57.7	4.10	7,023,600	55.8	2.18	2,719,736	63.4	9.25	4,303,864
1946 : 1945	1.06	.95	1.28	1.08	.92	1.28	1.02	1.03	1.28
South Atlantic:									
1941	65.6	3.88	10,982,765	64.6	2.61	5,663,491	69.0	8.05	5,319,274
1943	56.8	4.76	7,970,164	53.5	2.63	2,874,032	64.1	8.75	5,096,132
1945	57.4	4.84	8,350,393	53.2	2.48	2,803,119	67.5	9.28	5,547,274
1946	53.5	4.79	10,637,066	48.4	2.38	3,494,145	67.0	9.48	7,142,921
1946 : 1945	.93	.99	1.27	.91	.96	1.25	.99	1.02	1.29
Subtotal:									
1941	68.2	3.56	21,243,314	67.7	2.37	11,274,990	70.5	8.30	9,968,324
1943	56.9	4.43	14,312,430	53.8	2.45	5,408,970	65.1	8.73	8,903,469
1945	55.9	4.53	15,461,183	52.4	2.41	5,653,546	65.4	9.17	9,807,637
1946	54.9	4.44	19,656,508	51.1	2.29	7,058,238	66.1	9.39	12,598,270
1946 : 1945	.98	.98	1.27	.98	.95	1.25	1.01	1.02	1.28
Central regions:									
East North Central:									
1941	68.7	4.05	11,961,648	66.0	2.10	4,064,158	74.3	7.71	7,897,490
1943	62.3	4.74	8,381,988	54.3	2.18	2,148,969	76.5	7.98	6,233,019
1945	58.7	5.15	8,897,279	49.4	2.38	2,176,246	74.4	8.27	6,721,033
1946	57.2	4.81	10,081,218	50.5	2.17	2,685,088	70.6	8.59	7,396,130
1946 : 1945	.97	.93	1.13	1.02	.91	1.23	.95	1.04	1.10
East South Central:									
1941	64.8	3.13	3,303,537	64.1	2.70	2,546,285	70.5	6.58	757,252
1943	52.1	3.95	2,107,176	48.2	2.75	1,108,101	69.9	7.64	999,075
1945	47.0	4.01	2,136,854	43.4	2.72	1,108,602	63.8	8.23	1,028,252
1946	42.7	4.04	2,761,538	38.6	2.54	1,304,071	62.6	8.60	1,457,467
1946 : 1945	.91	1.01	1.29	.89	.93	1.18	.98	1.04	1.42
West North Central:									
1941	67.0	3.23	6,872,720	66.3	2.09	3,549,465	70.1	7.79	3,323,255
1943	63.3	3.30	4,994,660	63.2	2.06	2,448,202	63.6	7.76	2,546,458
1945	58.8	3.67	5,111,866	55.8	1.98	2,053,160	69.5	8.57	3,058,706
1946	55.1	3.72	6,585,211	52.2	1.97	2,582,219	65.2	8.74	4,002,992
1946 : 1945	.94	1.01	1.29	.94	.99	1.26	.94	1.02	1.31
West South Central:									
1941	60.6	2.99	5,427,182	59.6	2.23	3,144,368	64.5	5.60	2,282,814
1943	48.7	3.64	4,645,246	42.6	2.17	1,800,724	66.2	6.33	2,844,522
1945	42.6	3.79	4,165,546	37.1	2.54	1,808,167	58.2	6.05	2,357,379
1946	36.6	4.16	5,461,533	31.3	2.65	2,264,895	53.2	6.93	3,196,638
1946 : 1945	.86	1.10	1.31	.84	1.04	1.25	.91	1.15	1.36
Subtotal:									
1941	65.7	3.47	27,565,087	64.2	2.22	13,304,276	70.9	7.23	14,260,811
1943	57.4	3.95	20,129,070	52.6	2.21	7,505,996	70.3	7.47	12,623,074
1945	52.6	4.48	20,311,545	46.8	2.33	7,146,175	68.1	7.82	13,165,370
1946	48.6	4.24	24,889,500	43.4	2.26	8,836,273	63.7	8.23	16,053,227
1946 : 1945	.92	.95	1.23	.93	.97	1.24	.94	1.05	1.22
Western regions:									
Mountain:									
1941	62.6	3.39	2,795,644	61.6	2.46	1,755,379	69.7	9.45	1,040,265
1943	49.3	3.95	2,083,902	46.3	2.39	1,010,816	66.7	10.23	1,073,886
1945	45.8	4.63	2,374,966	41.5	2.51	950,677	74.5	10.66	1,424,289
1946	43.3	4.88	3,463,639	38.1	2.59	1,327,577	66.8	10.81	2,136,062
1946 : 1945	.95	1.05	1.46	.92	1.03	1.40	.93	1.01	1.50
Pacific:									
1941	68.8	5.13	7,132,737	65.2	2.14	2,151,962	80.4	12.84	4,980,775
1943	66.7	6.34	7,350,878	62.3	2.24	1,644,753	75.8	13.44	5,706,125
1945	65.4	7.05	12,216,924	58.1	2.57	2,435,956	77.3	12.49	9,780,968
1946	62.0	8.12	12,882,143	53.3	2.45	1,878,804	73.1	13.40	11,003,342
1946 : 1945	.95	1.15	1.05	.92	.95	.77	.95	1.07	1.12
Subtotal:									
1941	66.4	4.48	9,928,381	63.7	2.27	3,907,341	77.7	12.09	6,021,040
1943	60.0	5.60	9,434,780	55.3	2.30	2,655,569	73.8	12.80	6,779,211
1945	59.6	6.50	14,591,890	51.8	2.55	3,386,633	76.4	12.22	11,205,257
1946	54.7	7.12	16,345,782	46.0	2.51	3,206,378	71.8	12.90	13,139,404
1946 : 1945	.92	1.10	1.12	.89	.98	.95	.94	1.06	1.17
United States total:									
1941	66.7	3.64	58,736,782	65.4	2.29	28,486,607	74.6	8.23	30,250,175
1943	57.7	4.38	43,876,280	53.4	2.30	15,570,535	69.1	8.74	28,305,745
1945	55.1	4.84	50,364,618	49.6	2.40	16,186,354	69.2	9.31	34,178,264
1946	51.7	4.84	60,891,790	46.4	2.31	19,100,889	66.2	9.70	41,790,901
1946 : 1945	.94	1.00	1.21	.94	.96	1.18	.96	1.04	1.22

axles. *C* is a measure of load concentration and it is generally thought that a value of *C* greater than 750 is excessive.

The frequency of trucks and combinations with various values of *C* in 1946, 1945, and 1943 are shown in table 12 by census regions. Heavy load concentrations occur most often in the Pacific States, with the Middle Atlantic and East North Central States also having a high frequency. There has been a steady upward trend in this frequency throughout the country during the war years, although in comparing 1946 to the previous year, a slight over-all decline in heavy load concentrations is evident.

The American Association of State Highway Officials now recommends,[2] in lieu of a formula for the limitation of motor-vehicle weights, the use of a table of permissible weights based on the distance between the extremes of any group of axles. Table 13 shows data in relation to this and indicates, as does table 12, that overloading is most prevalent in the Pacific and Middle Atlantic regions, although in using this method of computing overloads there appears to be little variation among the years shown for the United States as a whole.

[2] *Policy concerning maximum dimensions, weights and speeds of motor vehicles to be operated over the highways of the United States*, adopted April 1, 1946, by the American Association of State Highway Officials; published by the association, 1946.

Table 10.—*Average maximum axle load of loaded trucks in the summer of 1946 and in corresponding periods of 1945 and 1943 and in the prewar period between 1936 and 1941*

Region	All trucks and combinations				Single-unit trucks				Truck combinations			
	1946	1945	1943	Prewar	1946	1945	1943	Prewar	1946	1945	1943	Prewar
	Pounds	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
New England	9,611	10,005	9,643	7,647	7,688	7,990	8,068	6,985	16,031	16,621	14,966	13,951
Middle Atlantic	11,054	11,584	10,440	7,858	8,924	9,346	8,725	6,779	16,979	16,768	16,135	13,551
South Atlantic	9,698	9,950	9,482	8,488	7,161	7,632	7,229	7,589	15,095	15,403	14,612	11,831
East North Central	11,047	11,678	10,915	8,290	8,273	8,595	8,335	6,806	14,949	14,797	14,466	12,090
East South Central	8,982	9,024	8,431	7,215	7,384	8,003	7,366	6,962	14,306	14,157	12,036	8,899
West North Central	9,228	9,300	9,060	7,593	7,117	7,212	7,391	6,565	14,585	14,633	13,488	11,594
West South Central	8,452	7,673	7,583	5,868	6,455	5,834	5,344	5,134	12,959	12,993	12,247	8,391
Mountain	9,035	8,858	8,313	6,051	7,109	7,110	6,990	5,444	14,669	14,118	14,176	11,284
Pacific	10,093	9,995	9,346	7,094	8,213	8,497	7,653	5,967	14,097	13,504	13,444	11,176
United States average	9,881	10,017	9,411	7,552	7,645	7,869	7,432	6,566	14,722	14,615	14,072	11,449

Table 11.—*Number of heavy axle loads and heavy gross weights per 1,000 loaded and empty trucks and combinations in the summers of 1946, 1945, 1943, and in the prewar period between 1936 and 1941*

Region	Number of heavy axle loads per 1,000 loaded and empty trucks and combinations of—												Number of gross weights per 1,000 loaded and empty trucks and combinations of—											
	18,000 pounds or more				20,000 pounds or more				22,000 pounds or more				30,000 pounds or more				40,000 pounds or more				50,000 pounds or more			
	1946	1945	1943	Prewar	1946	1945	1943	Prewar	1946	1945	1943	Prewar	1946	1945	1943	Prewar	1946	1945	1943	Prewar	1946	1945	1943	Prewar
New England	88	109	73	48	50	62	45	21	21	29	14	8	118	113	107	58	53	50	40	15	9	10	4	1
Middle Atlantic	151	149	128	40	85	75	75	18	42	33	28	7	149	163	159	62	80	77	59	17	25	18	17	3
South Atlantic	80	78	50	7	27	26	8	1	10	7	2	(1)	125	133	120	32	44	43	24	3	6	2	(1)	(1)
East North Central	93	74	62	14	27	10	15	4	6	5	4	2	194	233	199	65	80	72	48	13	36	28	16	5
East South Central	39	30	7	1	16	15	1	(1)	5	6	(1)	(1)	121	57	50	7	41	12	3	1	4	1	(1)	(1)
West North Central	49	41	32	5	13	8	4	(1)	3	1	1	(1)	112	114	105	34	45	33	35	2	14	11	3	(1)
West South Central	13	25	23	4	4	8	7	2	1	2	5	(1)	65	73	63	8	18	20	11	1	6	3	2	(1)
Mountain	46	44	33	5	16	12	8	3	5	3	5	2	92	98	83	20	53	54	46	7	31	28	28	3
Pacific	42	47	18	3	6	13	3	1	2	3	(1)	(1)	193	186	165	97	133	116	119	47	104	86	89	24
United States average	68	67	49	13	26	23	17	5	10	9	6	2	132	144	125	43	60	58	41	11	26	23	15	3

[1] Less than 5 per 10,000.

Table 12.—*Number of trucks with values of C in the gross weight formula in excess of various values, per 1,000 loaded and empty trucks and combinations in the summers of 1946, 1945, and 1943*

Region	Number of trucks and combinations with values of *C*—																	
	Over 650			Over 700			Over 750			Over 800			Over 850			Over 900		
	1946	1945	1943	1946	1945	1943	1946	1945	1943	1946	1945	1943	1946	1945	1943	1946	1945	1943
New England	39	33	33	22	18	14	11	9	7	6	4	3	1	2	1	1	1	(1)
Middle Atlantic	69	56	36	46	33	30	33	18	14	19	9	10	11	5	6	6	2	3
South Atlantic	16	20	10	7	9	3	5	5	2	3	3	(1)	1	1	0	1	1	0
East North Central	52	48	36	40	37	21	31	28	15	24	22	8	17	14	6	9	8	5
East South Central	6	5	3	2	1	1	2	(1)	(1)	1	(1)	0	(1)	(1)	0	0	(1)	0
West North Central	32	21	7	18	13	3	11	8	1	5	3	(1)	3	2	(1)	2	1	(1)
West South Central	8	9	8	5	3	4	5	1	2	4	1	1	2	(1)	1	1	(1)	1
Mountain	48	45	54	33	34	33	24	22	26	15	16	21	9	11	17	5	6	15
Pacific	126	135	131	107	112	111	81	89	88	46	59	60	15	22	31	3	8	17
United States average	43	45	31	31	33	18	22	24	15	14	17	9	7	8	6	4	4	4

[1] Less than 5 per 10,000.

Table 13.—*Number of trucks and combinations per 1,000 loaded and empty vehicles that exceeded the permissible motor-vehicle loads recommended by the A. A. S. H. O., and various percentages of overload, in the summers of 1946, 1945, and 1943*

Region	Number of trucks and combinations with overload																	
	Total overloaded			Over 5 percent			Over 10 percent			Over 20 percent			Over 30 percent			Over 50 percent		
	1946	1945	1943	1946	1945	1943	1946	1945	1943	1946	1945	1943	1946	1945	1943	1946	1945	1943
New England	17	13	8	10	6	5	5	4	3	1	1	1	(1)	(1)	0	0	0	0
Middle Atlantic	35	27	23	26	19	15	18	13	10	7	6	4	3	2	1	1	1	0
South Atlantic	5	7	2	3	4	1	3	3	(1)	1	2	(1)	(1)	0	0	(1)	0	0
East North Central	29	24	16	23	20	13	18	17	9	9	8	4	4	3	3	1	1	1
East South Central	2	(1)	(1)	1	(1)	(1)	1	(1)	0	(1)	(1)	0	0	(1)	0	0	0	0
West North Central	11	7	2	7	4	1	4	3	(1)	1	1	(1)	1	(1)	(1)	0	0	0
West South Central	5	2	2	4	1	2	3	1	1	1	1	1	1	(1)	0	(1)	0	0
Mountain	26	20	16	18	16	11	11	11	8	4	5	2	2	2	1	1	1	(1)
Pacific	79	67	100	47	49	71	20	24	51	3	4	19	(1)	0	3	0	0	0
United States average	22	22	21	15	16	15	10	10	10	3	4	4	1	(1)	1	(1)	(1)	(1)

[1] Less than 5 per 10,000.

Trends in Motor-Vehicle Travel, 1946

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH, PUBLIC ROADS ADMINISTRATION

Total motor-vehicle travel in the United States in 1946 amounted to more than 340 billion vehicle-miles, about evenly divided between urban and rural travel. This total was 2 percent greater than the previous record, established in 1941, and 37 percent above travel in 1945. The average vehicle traveled 9,958 miles in 1946, consuming 747 gallons of motor fuel at a rate of 13.32 miles per gallon.

CLASSIFIED estimates of motor-vehicle travel in the United States, previously published for the years 1936–45,[1] are here presented for the year 1946. Table 1 reports, for the various classes of motor vehicles, the estimates for 1946 of rural, urban, and total vehicle-miles traveled, average miles traveled and motor fuel consumed per vehicle, and average travel per gallon of motor fuel consumed.

The analysis on which these estimates are based was similar to that described in the previous report.[1] Because the volumes of motor-vehicle travel and motor-fuel consumption in 1946 were more nearly comparable to those in 1941 than any of the intervening years, 1941 was used as a base year in making the calculations.

Rural-road travel in 1946, reported in another article in this issue, was approximately one-half percent greater than in 1941. Highway use of motor fuel, however, was about 6 percent greater. Part of this apparent excess consumption of motor fuel is accounted for by the relative increase in the number of trucks and combinations, from 14.6 percent of the total motor-vehicle registration in 1941 to 17.1 percent in 1946. Part is due to the heavier loads carried by trucks and combinations during recent years, and an allowance for reduced average miles per gallon has been made in the calculations. Since these factors did not account for all of the apparent excess motor-fuel consumption, the indicated increase of 1946 urban travel over that in 1941 was greater than the increase of rural travel.

Total travel in 1946 was estimated as 340,655 million vehicle-miles—2 percent above the previous record set in 1941. Of the total, 170,606 million vehicle-miles were accounted for by rural travel and 170,049 million vehicle-miles by urban travel. While rural travel had increased only one-half percent over 1941, urban travel had risen almost 4 percent above that level.

Comparisons of 1946 travel with that of 1941 and 1945 are shown in table 2. The relative increase of 1946 rural travel over 1945 was much greater than that of urban travel. Rural travel suffered much greater set-backs during the war years and, in recovering, it naturally showed a greater percentage rise from 1945, during 8 months of which motor vehicles operated under wartime restrictions.

The relative increase of 1946 values over those of 1941, however, was greater for urban than for rural travel. A comparison of monthly rural traffic volumes in 1941 and 1946 shows that 1946 travel exceeded that of 1941 in January, February, March, April, September, and October, but was less in the usual peak-traffic months of June, July, and August, as well as in May, November, and December. Thus, while the level of rural traffic in 1946 was above that of 1941 during most of the winter, spring, and fall, summer traffic had not yet returned to normal. Probably the advanced age of millions of passenger cars somewhat deterred vacation and week-end touring; high costs of vacation accommodations may also have played a part. These factors would have had a minor effect on city travel, so it is not unnatural that the latter should display a more favorable comparison with 1941 levels than does rural travel.

The average miles traveled per vehicle in 1941 and 1946, for the two most important vehicle classes, were estimated as follows:

	1941	*1946*
Passenger cars	9,285	9,942
Trucks and combinations	10,739	9,615

Apparently inconsistent are the indications that the average mileage of trucks and combinations was less in 1946 than in 1941, while that of passenger cars was greater; and that passenger cars, which in past years traveled a lower annual mileage per vehicle than trucks and combinations, have exceeded the latter in mileage during 1946. A probable explanation lies in the 17.8 percent increase of truck and combination registrations in 1946 over those in 1945 while passenger car registrations increased only 9.4 percent. Factory sales figures indicate a large proportion of new truck sales in 1946 were made during the latter part of the year. This large influx of new

(*Continued on page 52*)

Table 1.—*Classified estimate of travel in the United States in calendar year 1946*

Vehicle type	Motor-vehicle travel			Number of registered vehicles [1]	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Rural travel	Urban travel	Total travel			Total [2]	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles per gallon*
Passenger vehicles:								
Passenger cars [3]	134,013	146,444	280,457	28,209	9,942	18,759	665	14.95
Busses:								
Commercial	1,413	1,987	3,400	80	42,500	680	8,500	5.00
School and nonrevenue	586	66	652	82	7,981	63	768	10.39
All busses	1,999	2,053	4,052	162	25,061	743	4,594	5.46
All passenger vehicles	136,012	148,497	284,509	28,371	10,028	19,502	687	14.59
Trucks and combinations	34,594	21,552	56,146	5,839	9,615	6,068	1,039	9.25
All motor vehicles	170,606	170,049	340,655	34,210	9,958	25,570	747	13.32

[1] These registrations differ from those given in Public Roads Administration table MV–1 for 1946 because of the following adjustments: (1) Approximate correction for defective classification in 6 States, as described in footnotes 7, 9, 11, 12, and 13 of that table; (2) inclusion of publicly owned vehicles, listed separately in table MV–1; (3) reduction of private and commercial truck registrations by 2.5 percent to allow for registration in more than 1 State; and (4) substitution of bus totals as estimated by the bus industry, to afford a complete segregation of commercial busses from school and nonrevenue busses and to allow for duplication because of registration in more than 1 State.

[2] Total highway use of motor fuel in 1946 is given as 25,649 million gallons in Public Roads Administration table G–21. For this analysis there was deducted from that total 79 million gallons estimated use by motorcycles (250 gallons per motorcycle).

[3] Including taxicabs.

Table 2.—*Increase in motor-vehicle travel, 1946 compared with 1941 and 1945*

Vehicle type	Percentage increase of travel—					
	1946 over 1941			1946 over 1945		
	Rural	Urban	Total	Rural	Urban	Total
Passenger vehicles	0.45	3.77	2.16	47.78	33.80	40.14
Trucks and combinations	.56	5.18	2.29	27.42	14.88	22.30
All vehicles	.47	3.95	2.18	43.15	31.07	36.85

[1] *Trends in motor-vehicle travel, 1936 to 1945*, by G. P. St. Clair; PUBLIC ROADS, vol. 24, No. 10; Oct.-Nov.-Dec. 1946.

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Trends in the Distribution of State Road-User Taxes, 1940–1947

BY THE DIVISION OF RESEARCH REPORTS
PUBLIC ROADS ADMINISTRATION

Reported by R. W. MEADOWS, Highway Economist
and W. R. McCALLUM, Associate Highway Economist

This article summarizes the recent trends and the current status of State–local highway finance relationships, with particular emphasis upon the distribution of State road-user tax revenues.

The large postwar highway programs, coupled with greatly increased construction costs, have intensified demands by both State and local groups for additional highway funds. In order to fill the more urgent needs, several States have been compelled to increase highway-user tax rates or to revise distribution formulas. This has brought about some misconceptions concerning the present allocation of these revenues. Actually, there does not appear to have been any significant change in the basic State–local distribution ratios in recent years. Both State and local roads have benefited from greater tax yields, with the State generally retaining the larger share of total revenue, for expenditures on State highway systems. Where revisions of the distribution formulas have occurred, increases in State or local shares have usually been accomplished by decreasing the portion of tax revenues formerly allocated to nonhighway purposes.

MOST States have shared a considerable part of their road-user revenues with their political subdivisions almost from the inception of special road-user taxes. The distribution of these revenues has always been a matter of lively interest. With the growing attention centered on highway needs and highway financing methods, much discussion has revolved around the changes that have been made in road-user revenue allocations in recent years.

Local groups, both rural and urban, have insistently presented their claims for a greater share of the revenue. At the same time, the need for an adequate system of main roads has at all times been so universally recognized that legislatures have been reluctant to increase the allocations for local roads and streets at the expense of delaying the development of the State arterial systems.

In order to clarify the State–local relationship, a study has been made of the 1940–47 trend in the distribution of road-user revenues to State highways, to local roads and streets, and also to nonhighway purposes. The 1940–46 portion of the study is based upon an analysis of the annual reports of State highway authorities to the Public Roads Administration, and the changes for 1947 are based upon the probable effects of legislation passed during the year. The 1940 and 1946 allocations are shown in detail in table 1.

CONCLUSIONS

An analysis of the 1940–1947 trend in highway financing methods indicates that while many States have in recent years provided additional funds for local roads and streets, this has rarely been done at the expense of revenues for State highways. In several cases local roads have received funds formerly allocated for nonhighway purposes, or have received a portion of new tax revenues. In a few instances they have received appropriations from State general fund surpluses. The States, however, have usually preserved the basic distribution ratios that were in effect in 1940, and where the ratios have been changed in favor of local roads, steps have been taken to assure that certain minimum revenues will be available to finance the postwar programs for improvement of State highways and secondary roads of more than local importance.

Both State highways and local roads have shared in an increase in State highway-user revenues, which in 1946 were 21 percent greater than the collections in 1940. Both are also receiving a greater portion of the total highway-user revenues: the State share increased from 58 percent of the amount distributed in 1940 to 64 percent in 1946, and the local share from 27 percent in 1940 to 30 percent in 1946. It is not the intention here to discuss the relative value of the rise in highway revenues, or the adequacy of the present highway-user tax structure. It should be obvious that increased revenues in recent years have been more than nullified by increased highway construction costs.

The increase in the State and local shares of highway-user revenues was accompanied by a decrease in the portion used for nonhighway purposes. This decrease has been the result of two factors: (1) The effectiveness of the campaign against allocation of user revenues to nonhighway purposes; and (2) increased appropriation of funds for highways out of general State revenue, offsetting statutory nonhighway allocations of highway-user revenue. Such appropriations, however, do not necessarily indicate a continuing policy.

COMPARISON OF 1940 AND 1946 ALLOCATIONS

By limiting this study to an evaluation of changes in the allocation of State funds for highways, some important factors in State–local highway relationships have necessarily been excluded. One of the most important of these is the effect of transferring mileage from the local road systems to State systems without providing a larger portion of total revenues to the State highway departments.[1] The roads thus taken over by the States are usually the most heavily traveled in the local systems, and therefore the most costly to improve and maintain. Also it may not be readily apparent that large amounts of State highway funds, while nominally expended in rural areas, are actually spent on heavily traveled roads adjacent to cities. Thus, while they are not actually expended within corporate limits, they are largely for the benefit of urban areas.

Another factor not included in this study is the control exercised by the States over funds allotted for local roads and streets. The degree of State control may influence the class of road upon which the money is spent.

While both highway revenues and highway construction were declining during the years 1942–45 as a result of wartime restrictions, the State legislatures gave little attention to revision of existing formulas for the distribution of funds. With the end of the war, however, the Federal-aid Highway Act of 1944 gave considerable impetus to expansion of highway construction on main State highways, on the more important secondary roads, and on arterial streets forming the major traffic channels within urban areas. At the same time, local governments renewed their demands for a greater share of State highway revenues.

By 1946 a few States had anticipated the postwar needs and demands by increasing highway-user tax rates, by reallocating the revenue from existing taxes, or by making appropriations for highways out of State general funds.

[1] The total mileage of the State highway systems, excluding county roads under State control in Delaware, North Carolina, Virginia, and West Virginia, increased from 434,000 miles in 1940 to 458,000 in 1946. The State-administered county roads in the four States mentioned increased from 114,000 miles to 119,000 miles during the same period.

Table 1.—*Distribution of State highway-user revenues, 1940 and 1946*

State	Total funds distributed		For highway purposes												For nonhighway purposes			
			For State highways [1]				For local roads and streets [2]				Total for highway purposes				Amount		Percent of total funds	
			Amount		Percent of total funds		Amount		Percent of total funds		Amount		Percent of total funds					
	1940	1946	1940	1946	1940	1946	1940	1946	1940	1946	1940	1946	1940	1946	1940	1946	1940	1946
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*			*1,000 dollars*	*1,000 dollars*			*1,000 dollars*	*1,000 dollars*			*1,000 dollars*	*1,000 dollars*		
Alabama	20,096	29,537	11,070	12,640	55.1	42.8	8,192	15,723	40.8	53.2	19,262	28,363	95.9	96.0	834	1,174	4.1	4.0
Arizona	5,838	9,618	4,367	7,344	74.8	76.4	1,471	2,274	25.2	23.6	5,838	9,618	100.0	100.0				
Arkansas	14,756	21,263	13,329	18,434	90.3	86.7	1,427	2,774	9.7	13.0	14,756	21,208	100.0	99.7		55		.3
California	77,254	107,702	40,860	50,442	52.9	46.8	22,405	47,758	29.0	44.4	63,265	98,200	81.9	91.2	13,989	9,502	18.1	8.8
Colorado	10,948	14,169	7,387	9,600	67.5	67.8	3,561	4,569	32.5	32.2	10,948	14,169	100.0	100.0				
Connecticut	17,244	18,738	14,017	10,251	81.3	54.7	3,226	8,487	18.7	45.3	17,243	18,738	100.0	100.0	1			
Delaware	3,420	3,735	3,392	3,515	99.2	94.1	2				3,394	3,515	99.2	94.1	26	220	.8	5.9
Florida	34,757	49,925	23,478	25,859	67.5	51.8	281	6,068	.9	12.1	23,759	31,927	68.4	63.9	10,998	17,998	31.6	36.1
Georgia	24,609	33,975	15,456	22,258	62.8	65.5	5,337	5,897	21.7	17.4	20,793	28,155	84.5	82.9	3,816	5,820	15.5	17.1
Idaho	6,232	9,254	3,843	6,867	61.7	74.2	2,389	2,387	38.3	25.8	6,232	9,254	100.0	100.0				
Illinois	64,322	68,473	33,798	38,278	52.5	55.9	23,539	30,195	36.6	44.1	57,337	68,473	89.1	100.0	6,985		10.9	
Indiana	36,282	37,317	20,623	21,505	56.8	57.6	13,944	15,225	38.4	40.8	34,567	36,730	95.2	98.4	1,715	587	4.8	1.6
Iowa	27,244	27,960	19,238	18,699	70.6	66.9	8,006	9,261	29.4	33.1	27,244	27,960	100.0	100.0				
Kansas	15,400	21,024	11,765	17,056	76.4	81.1	3,635	3,968	23.6	18.9	15,400	21,024	100.0	100.0				
Kentucky	19,396	25,808	14,953	19,964	77.1	77.4	2,606	5,844	13.4	22.6	17,559	25,808	90.5	100.0	1,837		9.5	
Louisiana	22,257	28,966	16,009	21,146	71.9	73.0		575		2.0	16,009	21,721	71.9	75.0	6,248	7,245	28.1	25.0
Maine	10,045	11,338	8,745	10,511	87.1	92.7	1,158	827	11.5	7.3	9,903	11,338	98.6	100.0	142		1.4	
Maryland	17,713	20,897	10,353	11,006	58.5	52.7	6,025	8,271	34.0	39.6	16,378	19,277	92.5	92.3	1,335	1,620	7.5	7.7
Massachusetts	26,846	27,119	11,180	17,444	41.6	64.3	14,403	9,675	53.7	35.7	25,583	27,119	95.3	100.0	1,263		4.7	
Michigan	55,566	59,981	27,499	28,871	49.5	48.1	28,067	31,110	50.5	51.9	55,566	59,981	100.0	100.0				
Minnesota	27,843	31,628	21,490	24,007	77.2	75.9	6,106	7,382	21.9	23.3	27,596	31,389	99.1	99.2	247	239	.9	.8
Mississippi	14,789	20,415	7,173	10,088	48.5	49.4	7,616	10,327	51.5	50.6	14,789	20,415	100.0	100.0				
Missouri	23,527	25,148	23,439	25,148	99.6	100.0					23,439	25,148	99.6	100.0	88		.4	
Montana	6,933	7,485	5,448	6,072	78.6	81.1	1,485	1,394	21.4	18.6	6,933	7,466	100.0	99.7		19		.3
Nebraska	14,227	18,217	6,412	8,604	45.1	47.2	5,493	6,535	38.6	35.9	11,905	15,139	83.7	83.1	2,322	3,078	16.3	16.9
Nevada	2,001	2,868	2,001	2,868	100.0	100.0					2,001	2,868	100.0	100.0				
New Hampshire	6,514	6,860	5,735	6,230	88.0	90.8	779	630	12.0	9.2	6,514	6,860	100.0	100.0				
New Jersey	44,872	45,703	23,086	30,040	51.5	65.7	10,296	15,046	22.9	32.9	33,382	45,086	74.4	98.6	11,490	617	25.6	1.4
New Mexico	6,510	9,476	5,627	8,399	86.4	88.6	267	338	4.1	3.6	5,894	8,737	90.5	92.2	616	739	9.5	7.8
New York	123,844	120,558	32,345	102,374	26.1	84.9	27,517	18,184	22.2	15.1	59,862	120,558	48.3	100.0	63,982		51.7	
North Carolina	35,629	50,225	23,977	25,610	67.3	51.0	10,519	22,941	29.5	45.7	34,496	48,551	96.8	96.7	1,133	1,674	3.2	3.3
North Dakota	4,956	5,281	3,414	3,640	68.9	68.9	1,539	1,641	31.0	31.1	4,953	5,281	99.9	100.0	3		.1	
Ohio	77,519	92,345	30,445	39,517	39.3	42.8	33,785	43,659	43.6	47.3	64,230	83,176	82.9	90.1	13,289	9,169	17.1	9.9
Oklahoma	20,251	39,592	12,605	29,380	62.2	74.2	7,414	10,212	36.6	25.8	20,019	39,592	98.8	100.0	232		1.2	
Oregon	15,380	22,568	12,818	19,005	83.3	84.2	2,423	3,563	15.8	15.8	15,241	22,568	99.1	100.0	139		.9	
Pennsylvania	99,184	99,595	71,341	83,184	71.9	83.5	12,019	16,383	12.1	16.4	83,360	99,567	84.0	99.9	15,824	28	16.0	.1
Rhode Island	6,893	7,070	2,895	3,640	42.0	51.5	56	59	.8	.8	2,951	3,699	42.8	52.3	3,942	3,371	57.2	47.7
South Carolina	15,902	21,278	13,368	17,634	84.0	82.9	2,317	2,967	14.6	13.9	15,685	20,601	98.6	96.8	217	677	1.4	3.2
South Dakota	6,486	7,200	4,512	5,513	69.6	76.6	1,867	1,626	28.8	22.6	6,379	7,139	98.4	99.2	107	61	1.6	.8
Tennessee	26,390	38,758	12,528	23,351	47.5	60.2	5,930	9,178	22.5	23.7	18,458	32,529	70.0	83.9	7,932	6,229	30.0	16.1
Texas	66,976	92,069	37,841	55,438	56.5	60.2	17,549	17,189	26.2	18.7	55,390	72,627	82.7	78.9	11,586	19,442	17.3	21.1
Utah	4,994	7,175	4,084	5,465	81.8	76.2	873	1,073	17.5	14.9	4,957	6,538	99.3	91.1	37	637	.7	8.9
Vermont	5,318	5,755	2,810	3,124	52.8	54.3	2,480	2,608	46.7	45.3	5,290	5,732	99.5	99.6	28	23	.5	.4
Virginia	26,175	36,861	17,969	17,762	68.7	48.2	8,197	19,022	31.3	51.6	26,166	36,784	100.0	99.8	9	77		.2
Washington	20,357	28,644	10,361	14,840	50.9	51.8	8,924	13,804	43.8	48.2	19,285	28,644	94.7	100.0	1,072		5.3	
West Virginia	17,214	20,090	14,259	16,578	82.8	82.5	2,955	3,512	17.2	17.5	17,214	20,090	100.0	100.0				
Wisconsin	34,893	42,849	11,037	28,029	31.6	65.4	11,016	10,993	31.6	25.7	22,053	39,022	63.2	91.1	12,840	3,827	36.8	8.9
Wyoming	3,553	5,794	2,864	4,694	80.6	81.0	689	1,100	19.4	19.0	3,553	5,794	100.0	100.0				
Dist. of Columbia	5,034	5,992					4,779	5,542	94.9	92.5	4,779	5,542	94.9	92.5	255	450	5.1	7.5
Total	1,274,389	1,544,298	733,246	991,924	57.5	64.2	344,564	457,796	27.1	29.7	1,077,810	1,449,720	84.6	93.9	196,579	94,578	15.4	6.1

[1] Includes $629,000 allotted for county roads under State control in Delaware in 1940. Segregation is not available for 1946.
[2] Includes amounts allotted for county roads under State control in North Carolina, Virginia, and West Virginia. Similar allotments in Delaware are included with State highways. (See note 1.)

Increased highway-user tax rates.—Five States increased their motor-fuel tax rates during 1945 or 1946. In Idaho, Kansas, Oklahoma, and Virginia, the increase was made to assure adequate resources to match the Federal funds provided for the three Federal-aid programs. In none of these four States was the additional revenue intended to be used for roads of less importance than those in the Federal-aid systems. In Iowa, however, the motor-fuel tax was increased to provide funds for local roads and streets, without reference to the Federal program.

Reallocations of existing tax revenues.—In only one State, Alabama, was there a reallocation of existing taxes to provide a smaller share for State highways and a larger share for local roads. In 1943 the State decreased its motor-fuel tax allotment for State highways from 3 cents to 2 cents, and increased the amount for local roads and streets from 3 cents to 4 cents. The 1 cent taken away from State highways was to be used to match 1 cent of the county share to improve selected county roads, including those forming a part of the Federal-aid secondary system.

In Connecticut, Kentucky, North Carolina, Pennsylvania, and Virginia, where allocations are not based entirely on a percentage of revenues, appropriations for local roads and streets out of the State highway fund have increased at a greater rate than have the revenues collected. In Kentucky and Pennsylvania, the increases for local roads have been largely offset by a discontinuance of allocations for nonhighway purposes. In Connecticut, a 1947 increase in the motor-fuel tax rate may restore the prewar relation between appropriations for State highways and those for local roads. In North Carolina and Virginia, the large increase in appropriations for local roads under State control, made in 1945 and 1946, was partly to match Federal-aid secondary funds and partly to complete war-deferred improvements. In Massachusetts, the appropriations for local roads decreased during this period.

In Kentucky, Pennsylvania, and Wisconsin, basic changes in the distribution of highway-user revenues or in the handling of highway accounts resulted in the allocation for highway purposes of funds formerly used for nonhighway purposes. Nonhighway allocations in Kentucky decreased from 10 percent of total highway-user revenues in 1940 to none in 1946; in Pennsylvania the change was from 16 percent to less than 1 percent; and in Wisconsin from 37 percent to less than 9 percent. The immediate effect of the reallocation in Wisconsin was to provide additional funds for State highways, while in Kentucky and Pennsylvania it served to offset an increase in appropriations for local roads.

Appropriations from State general funds.—Many States accumulated large surpluses of general revenues during the war years. These surpluses have been used by some States to increase funds for the State and local highway programs. To some extent they have also been used to defer a demand for an increase in highway-user tax rates or a reallocation of existing taxes.

Between 1943 and 1946, considerable amounts were provided in this manner for local roads and streets by California, Illinois, and Washington, and for State highways by Louisiana, Ohio, and Oklahoma. Mississippi

and West Virginia appropriated general revenues for both State and local roads. Only in West Virginia, where all county roads are under State control, were the appropriations the result of a continuing policy that was in effect in 1940.

Five other States—Delaware, Georgia, New Jersey, New York, and Rhode Island—regularly make appropriations for highways out of State general funds. These States, however, do not segregate highway revenues in a highway fund: they place all revenues in the State general fund and appropriate therefrom for all purposes, including highways. There has been no significant change in the ratio of amounts appropriated for State highways and for local roads and streets during recent years in these "one-fund" States. In New York, where highway appropriations in prior years were considerably less than the total highway revenues, the appropriations for 1945 and 1946 greatly exceeded such revenues.

In those States where appropriations for highways are made out of State general funds, while at the same time highway-user revenues are allocated by law to nonhighway purposes, the general fund appropriations, in effect, compensate highway funds partly or fully for the loss of highway-user revenues. This factor has been taken into consideration in determining the highway and nonhighway allocations in table 1.

EFFECTS OF 1947 LEGISLATION

The first general postwar legislative sessions in 1947 were confronted with insistent demands for additional funds for both State highways and local roads and streets. Rapidly increasing highway costs, together with the expanding highway programs, made it necessary for several States to seek additional revenues by increasing highway-user tax rates. In a few others the revenue from existing taxes was reallocated to provide additional funds for local roads and streets. Other States continued to appropriate funds for highways out of general revenues, but the full effect of these appropriations cannot yet be determined.

Increased highway-user tax rates.—Seven States—California, Colorado, Connecticut, Maine, Maryland, Rhode Island, and Vermont—increased their motor-fuel taxes during 1947. Several States also increased their motor-vehicle fees. From available information, it appears that in only one of these States, Colorado, has there been any basic change in the proportions of total highway-user revenues to be allotted to State highways and to local roads and streets. Under the previous 4-cent motor-fuel tax, Colorado allotted 27 percent to the counties. With an increase of 2 cents in the tax and some changes in its distribution, the counties and cities together will now receive approximately 37 percent of total motor-fuel revenues.

California and Maryland revised their distribution formulas when they increased highway-user tax rates, but the net result appears to be that State–local sharing was not materially changed. Nonhighway allocations were eliminated in Maryland and decreased in California.

Reallocations of existing tax revenues.—Eight States reallocated their highway-user revenues to give a greater share to local governments for roads and streets. In four of these the revisions affect the funds available for State highways: Oregon increased the cities' share of highway-user revenues from 5 percent to 10 percent and the counties' share from 15.7 percent to 19 percent, while Arkansas, Indiana, and Wisconsin allotted additional funds for local roads and streets provided the State receives certain guaranteed minimum amounts. The remaining four States in this group—Nebraska, New Mexico, Oklahoma, and Utah—increased their allotments to local roads and streets by decreasing or eliminating former nonhighway allocations.

(*Continued from page 49*)

vehicles, which were driven relatively low mileages during 1946, decreased the average travel per vehicle even though the average for trucks and combinations operated for the full 12-month period may have been as great as in 1941, or greater. The similar effect for passenger cars was of much less magnitude because the relative increase in numbers was much lower. Probably, since the number of trucks registered in 1946 was 14 percent greater than in 1941, the demand for new trucks came much nearer to being satisfied than that for passenger cars. The unslaked demand for the latter is reflected in an average mileage per car that is probably greater than would have been found if a normal number had been available for use.

The annual consumption of motor fuel per vehicle has risen well above prewar levels, from 694 gallons in 1941 to 747 in 1946. However, the 13.32 miles per gallon averaged by all vehicles in 1946 is somewhat below the 13.75 average of 1941. This is due chiefly to the following causes: (1) Trucks and combinations formed a larger percentage of the total motor-vehicle registration in 1946; (2) the average operating gross loads of trucks and combinations have increased substantially; and (3) because the average age of passenger cars was much greater than formerly, and their relative use of rural roads as compared with their urban travel had not quite returned to normal, it is probable that the average miles per gallon of passenger cars was somewhat below the prewar level.

The Effect of Petroleum-Like Constituents on Road Tars

and

A Test for Determining the Homogeneity of Road Tars

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by R. H. LEWIS, Senior Chemist and W. J. HALSTEAD, Associate Chemist

Differences in behavior, during construction, of road tars meeting the same specifications have been traced to the presence in the tars of petroleum-like constituents consisting of unsulfonatable hydrocarbons. In this investigation it was found that the unsulfonatable hydrocarbons have a flocculating effect on the tar which increases as the percentage of unsulfonatable hydrocarbons increases until a separation of the blend into two phases results. The progressive flocculation up to the point of separation can be detected only by microscopic examination.

In the course of the investigation, a new test was devised to measure the relative compatability of unsulfonatable hydrocarbons and tars. It involves adding increments of a petroleum distillate to the tar and noting the "separation point"—the percentage of diluent at which the blended materials separate into two phases. The test indicates the relative amount of unsulfonatable hydrocarbons in tars from the same source, but cannot be used as an exact determination of these constituents in lieu of the sulfonation-index test. The new separation-point test will be useful as a rapid laboratory check on the uniformity of tar shipments from the same source.

DIFFERENCES in behavior of road tars meeting the same specification have frequently been noted by engineers using these materials in road construction. Recent studies to determine the cause for such variations in behavior point to the presence of petroleum-like constituents in the tar. It is well known that petroleum products and tars do not blend completely in all proportions, and thus there is a tendency toward incompatibility between these petroleum-like constituents and the normal constituents of the tar.

The presence of the petroleum-like constituents in tar is occasioned in two ways: First, by the use, as bases or fluxes, of water-gas tars that through improper processing contain a considerable amount of the original petroleum oil which has not been cracked or altered in the carburation process; and second, by the unintentional mixture in shipment, through failure to steam-clean a tank car or truck that had contained petroleum products before loading it with tar.

Since the usual tests made on road tars give no indication of the relative compatibility of petroleum-like constituents and tar, a new test has been developed to measure this condition. Briefly, this test is made by adding a petroleum distillate to measured portions of tars, thoroughly mixing the components, and noting the percentage of diluent at which the blended materials separate into two phases. The percentage of diluent required to bring about a separation is termed the "separation point."

Since the separation point represents an end point only and the visual observations give no indication of the intermediate effects of lesser additions of the petroleum distillate, further investigations were conducted in which the progressive changes in the tar structure produced by increasing increments of the petroleum-like materials were observed under a microscope. In addition, the effect of these changes on the kinematic viscosity of the tar blends was also studied.

The data obtained in each of the phases of this study are closely interrelated and the report has been arranged so as to give not only the development and significance of the separation-point test but also to present as complete a picture as possible of the changes produced in a tar by the presence of any percentage of petroleum-like material from zero up to the separation point.

The problems discussed in this paper are generally the same as those previously discussed in a report on the sulfonation-index test,[1] and a brief review of the basic chemical groups described in that paper and of the general conclusions drawn at that time are given here in order that the principles involved in this investigation may be fully understood.

The phenomenon of separation exhibited by blends of petroleum products and tars is not caused by any inherent difference in petroleum and tar, but rather because of the partial incompatibility of the major constituents usually found in these materials.

The major proportion of the constituents of most tars are hydrocarbons of the aromatic group, while uncracked petroleum products contain chiefly those of the paraffinic and naphthenic groups. Cracked petroleum products contain high percentages of the olefinic group, also. However, both tars and petroleum products may contain some percentage of all these groups; and the percentage of any group that may be present in either type of material will vary, depending on the source of the crude and the manufacturing process used. It is the paraffinic and naphthenic hydrocarbons that have been termed the petroleum-like constituents and whose presence in a tar is considered undesirable.

The maximum amounts of the paraffinic and naphthenic groups permitted in a road tar are controlled by the recently adopted specification requirements for the sulfonation index. This index represents the amount of material in the tar distillate that is unaffected when the distillate is treated with a very strong (98.6 percent) sulfuric acid. These are the paraffinic and naphthenic hydrocarbons, since the conditions of the test are so controlled that all other groups present will be oxidized or converted into sulfonic acids and dissolved in the excess sulfuric acid present in the reacting mixture.

In the report on the sulfonation index, it was shown that the presence of the unsulfonatable (paraffinic and naphthenic) hydrocarbons has an adverse effect on the binding and weathering properties of the tar. In that investigation, however, no attempt was made to evaluate the effect of these constituents on the structure of the tar. In this investigation the effect of the paraffinic and naphthenic constituents on the tar structure was of primary interest.

CONCLUSIONS

The data presented in this report are concerned chiefly with the effect on tar of the unsulfonatable hydrocarbons which have distillation characteristics similar to those of the distillate from RT–2 to RT–4 grade road tars.

[1] *The sulfonation index test for road tars*, by R. H. Lewis and W. J. Halstead; PUBLIC ROADS, vol. 23, No. 7; Jan.-Feb.-Mar. 1943.

Some differences in behavior for the various types of base materials were obtained which are believed to be chiefly caused by differences in viscosity. It is probable that unsulfonatable materials which distill at a lower or higher range in temperature will show additional variations in behavior. It is believed, however, that the fundamental cause of the phenomena observed is the same in all cases. As a result of this study the following conclusions, based on the behavior of the RT–2 to RT–4 grades, may be drawn:

1. The unsulfonatable hydrocarbons present in a tar blend have a flocculating effect on the dispersed phase of the tar. This effect increases as the percentage of unsulfonatable hydrocarbons increases until a separation into two phases results.

2. Tests on filtered tars showed that, in addition to this flocculation of the dispersed phase, large amounts of unsulfonatable hydrocarbons caused the development of incompatibility of certain constituents originally soluble in the continuous phase.

3. No evidence of the flocculation of the dispersed phase can be seen with the unaided eye until the separation point is reached. The progressive development of this flocculation for increasing percentages of Diesel oil can be seen by microscopic examination.

4. The separation-point test is a means of measuring the relative compatibility of the constituents in a tar blend, and can be used as a rapid check in the laboratory to determine the uniformity of shipments from the same source.

5. The separation-point test gives an indication of the relative amount of unsulfonatable hydrocarbons in tars from the same source, but it cannot be used as an exact determination of these constituents in lieu of the sulfonation-index test.

6. No attempt has been made to correlate directly the results of the separation-point test with service behavior or laboratory tests designed to show bonding strength or other road building properties. Such correlations may be impossible to obtain because of the many complex factors that affect the quality of a road tar. However, it is believed that the compatibility of the various constituents in a road tar is an important one of these factors, and the separation-point test is a simple means of securing a relative measure of this property.

PETROLEUM DISTILLATE CHARACTERISTICS

There are a number of petroleum distillates available, all of which will give similar results when used as the diluent in determining the separation point. In this investigation, preliminary studies were conducted with four types of petroleum oil: A close-cut paraffinic distillate in the boiling range of kerosene, Bunker–B fuel oil, No. 2 grade fuel oil, and Diesel oil. It was found that separation occurred with lower percentages of the paraffinic distillate and Bunker–B fuel oil than with the No. 2 grade fuel oil or Diesel oil, the latter two giving comparable results.

For the purpose of the investigation it was desirable to use a material that had the same general distillation characteristics as the tar distillate from the more fluid grades of tar, and also one in which the unsulfonated hydrocarbons would be chemically similar to the unsulfonated materials in the tars.

Preliminary tests on several petroleum distillates showed Diesel oil to have these desirable characteristics. Table 1 compares the characteristics of the Diesel oil and its unsulfonated residue with a typical distillate from a RT–2 grade tar, and with a composite sample of the unsulfonated residues from numerous samples of tars that had formerly been tested in the laboratory. The last-mentioned material was obtained by accumulating the unsulfonated residues from all the tars tested in the laboratory over a period of several years. The residues were combined, washed free of acid, separated from the water, and dried with anhydrous sodium sulfate.

Table 1 shows the specific gravity, the index of refraction, and distillation characteristics for each of the distillates. These distillation data are plotted in figure 1. It is shown by these results that the unsulfonated residue from the tar distillates and the unsulfonated residue from the Diesel oil have very nearly the same boiling range, specific gravity, and index of refraction, and therefore must be chemically similar. It is also shown that the distillation curve of the Diesel oil itself does not vary greatly from that of the tar distillate.

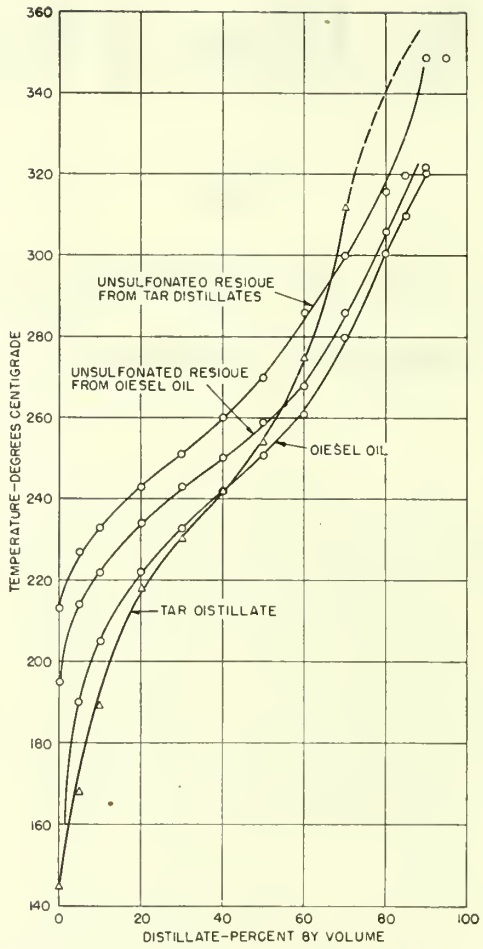

Figure 1.—Comparison of distillation properties of Diesel oil, a tar distillate, and their unsulfonated residues.

Table 1.—*Comparison of test characteristics of various types of distillates*

	Diesel oil	Unsulfonated residue from Diesel oil	RT–2 tar distillate (to 355° C.)	Unsulfonated residue from tar distillate (to 355° C.)
Specific gravity at 25°/25° C	0.842	0.808	0.995	0.827
Index of refraction at 25° C	1.469	1.447	1.587	1.452
Distillation characteristics: [1]				
Initial boiling point, degrees C	125	195	145	213
Vapor temperature at which:				
5 percent boiled off, degrees C	190	214	168	227
10 percent boiled off, do	205	222	189	233
20 percent boiled off, do	222	234	218	243
30 percent boiled off, do	233	243	230	251
40 percent boiled off, do	242	250	242	260
50 percent boiled off, do	251	259	254	270
60 percent boiled off, do	261	268	275	286
70 percent boiled off, do	280	286	312	300
80 percent boiled off, do	301	306	[2]	316
85 percent boiled off, do	310	320	[2]	
90 percent boiled off, do	321	322	[2]	349
95 percent boiled off, do				349
End point, do	321	322	367	349
Volume distillate at end point, percent	92	90	93	95

[1] Distillation made using 20 milliliters in a 50-milliliter Engler distilling flask, with water-cooled condenser.
[2] Readings not obtained because of solids in the condenser.

For further identification of the Diesel oil, the results of a number of standard tests are given below:

Specific gravity at 25°/25° C	0.842
Density at 35° C, gms. per cm³	.832
Volume distillation characteristics (A. S. T. M. test method D158–41):	
Initial boiling point, degrees C	120
Vapor temperature at which:	
10 percent boiled off, degrees C	194
20 percent boiled off, do	211
30 percent boiled off, do	229
40 percent boiled off, do	238
50 percent boiled off, do	248
60 percent boiled off, do	257
70 percent boiled off, do	268
80 percent boiled off, do	282
90 percent boiled off, do	302
End point, do	318
Volume distillate at end point, percent	94
Residue at end point, do	5
Weight distillation characteristics (A. S. T. M. test method D 20–30):	
Distillate to 300° C, percent	87.2
Distillate 300–318° C, do	8.5
Residue, do	3.6
Sulfonation index:	
Distillate to 300° C	66.3
Distillate 300–318° C	6.1
Volume of Diesel oil unsulfonated, percent	64

PROCEDURE FOR DETERMINING THE SEPARATION POINT OF TARS

Weigh 10 milliliters of tar (weight calculated from density at 25° C.) into a flat 2-ounce ointment can and add Diesel oil from a burette in increments so that each increment will increase the percentage by volume of Diesel oil in the final blend by 2.5 percent.[1] The sample shall be stirred thoroughly after each addition of Diesel oil [2] [3] and examined for evidence of separation. When a definite precipitation or separation remains after several minutes of stirring, this sample shall be set aside and Diesel oil added to other 10-milliliter portions of the tar so that the percentages of Diesel oil in the blends are 2.5, 5.0, 7.5, etc. percent [4] lower than that at which separation was obtained with the initial sample. The blended samples shall be allowed to remain undisturbed overnight and then examined for separation. Their condition shall then be noted on the following basis:

NS (No Separation)—The appearance and flow properties of a normal tar remain.

VSS (Very Slight Separation)—No definite separation can be detected, but the material does not flow as a normal tar.

SS (Slight Separation)—Some evidence of separation, gelling, or precipitation can be detected initially, but after thorough stirring it is difficult to see.

DS (Definite Separation)—Unmistakable separation, gelling, or precipitation can be detected even after thorough stirring.

The lowest percentage, by volume, of Diesel oil that causes slight separation (*SS*) shall be reported as the separation point. Sometimes the point of slight separation cannot be determined with precision and in such cases the percentage of Diesel oil that causes definite separation (*DS*) shall be reported as the separation point.

NOTES

[1] The milliliters of Diesel oil added to a 10-milliliter sample of tar to give the required percentages of diluent in mixtures are as follows:

Percentage of diluent	Volume added (ml.)	Percentage of diluent	Volume added (ml.)
2.5	0.26	27.5	3.79
5.0	.53	30.0	4.29
7.5	.81	32.5	4.81
10.0	1.11	35.0	5.39
12.5	1.43	37.5	6.00
15.0	1.77	40.0	6.67
17.5	2.12	42.5	7.40
20.0	2.50	45.0	8.18
22.5	2.90	47.5	9.05
25.0	3.33	50.0	10.00

[2] Viscous samples, or those that are difficult to blend, may be warmed on a steam bath. The temperature should not be allowed to exceed 60° C. and the sample should be covered while on the steam bath. When it is known that the separation point will be relatively high, increments larger than 2.5 percent may be added until the end point is approached.

[3] All stirring should be done with a spatula, since a slight precipitation is more readily seen when the tar is allowed to flow over a flat surface.

[4] Usually the separation point is 2.5 or 5.0 percent lower than the point of definite separation determined on the first samples so that three portions of the tar are sufficient to determine the separation point after settling overnight. However, if tests on three samples fail to give the desired range, additional tests must be made until the lowest point of slight separation is determined.

After considerable experimentation with various size samples, various temperatures, and various methods of blending, the laboratory procedure for making the test for separation point was standardized as described in a separate statement on this page.

A check on the precision of the test was conducted by having three operators determine the separation point of a small group of specially prepared samples. The results of these tests are shown in table 2. In most cases the same observations were recorded, and in all cases except for sample *3–X*, the same separation point was reported by all operators. For sample *3–X*, operator 2 reported the separation point as 35.0 percent, while operators 1 and 3 reported 32.5 percent. Since operators 2 and 3 had no instructions other than those in the written procedure, it was concluded that personal variations in determining the points at which separation occurs will not be great and close agreement should be obtained by different operators or laboratories when the same diluent is used.

TARS AND FLUXES FROM KNOWN SOURCES USED

Through the courtesy of one of the tar manufacturers, five crude base tars from different sources and four different types of fluxing material were obtained for use in this investigation. The test characteristics of these materials are shown in table 3.

Sample *A*, the coke-oven tar, is of a type that normally gives very good service when used as a road tar.

Sample *B*, the horizontal-retort tar, is of a type that is produced only in small quantities in the United States and it is rarely used in road-tar manufacture.

Sample *C*, the vertical-retort tar, is usually blended with other types of material, such as a heavy water-gas tar, when used in a road tar. It is generally considered unsatisfactory if used alone.

Sample *D*, the heavy water-gas tar, is one of a very large class of tars that exhibit considerable variation in properties. These tars are widely used as base materials for road-tar manufacture.

Sample *E* is an extremely viscous water-gas tar that would probably be blended with less viscous bases if used in a road tar.

Sample *F* represents a low-boiling coal-tar distillate, and sample *G* a medium-boiling distillate. Both these materials are satisfactory fluxes when used in a balanced blend.

Sample *H*, the low sulfonation-index water-gas tar, is rated as very good fluxing material for any type base.

Sample *I* is reported to be the highest sulfonation-index tar available to the manufacturer who supplied the samples, yet it is a satisfactory flux when used in limited quantities.

The tar acids were removed from the crude coal tars, samples *A*, *B*, and *C*, by distilling to 270° C. and treating the distillate with sodium hydroxide. The residual tar oils were then separated from the caustic, washed, dried with anhydrous sodium sulfate, and recombined with the distillation residue. The tests resulting in the data shown in table 3, and all other tests, were conducted on these recombined samples. Samples *D* and *E* were not treated but were used as received.

SPECIAL PRESSURE FILTER USED

In order to trace the effect of the unsulfonatable hydrocarbons on the tar after removal of the free carbon, a portion of each base tar was filtered. Tests with these filtered tars

Table 2.—*Comparison of the separation points of selected samples, determined by different operators*

Identification No.	Volume of Diesel oil in blend	Condition of blend [1] reported by operator—			Separation point [2] reported by operator—		
		1	2	3	1	2	3
	Percent						
1–X	2.5	*NS*	*NS*	*NS*	5.0	5.0	5.0
	5.0	*SS*	*SS*	*SS*			
	7.5	*DS*	*DS*	*DS*			
	10.0	*DS*	*DS*	*DS*			
2–X	2.5	*DS*	*SS*	*SS*	2.5	2.5	2.5
	5.0	*DS*	*DS*	*DS*			
	7.5	*DS*	*DS*	*DS*			
3–X	30.0	*NS*	*NS*	*NS*	32.5	35.0	32.5
	32.5	*SS*	*VSS*	*SS*			
	35.0	*DS*	*DS*	*DS*			
4–X	22.5	*NS*	*NS*	*NS*	27.5	27.5	27.5
	25.0	*VSS*	*VSS*	*VSS*			
	27.5	*DS*	*DS*	*DS*			
5–X	10.0	*NS*	*NS*	*NS*	15.0	15.0	15.0
	12.5	*NS*	*NS*	*VSS*			
	15.0	*SS*	*SS*	*SS*			
6–X	2.5	*DS*	*DS*	*DS*	2.5	2.5	2.5
	5.0	*DS*	*DS*	*DS*			
	7.5	*DS*	*DS*	*DS*			

[1] *NS*=no separation; *VSS*=very slight separation; *SS*=slight separation, *DS*=definite separation.
[2] As represented by percentage volume of Diesel oil in blend.

were conducted in parallel with tests on the unfiltered tars, except that the viscosity determinations of the Diesel-oil blends with the filtered tars were not made. The filtration was accomplished in a special pressure filter without the use of any solvent or diluent. This filter was designed and constructed according to the principle of that sketched by Volkmann, Rhodes, and Work.[2]

The filter medium is an alundum extraction shell, 34 millimeters in diameter and 100 millimeters high. Experiments showed that the coarse or medium porosity shells gave equally satisfactory results. Air pressure was maintained manually at 50 to 60 pounds per square inch. The water bath was heated electrically and thermostatically controlled at the desired temperature. The flask receiving the filtrate was immersed in ice water to reduce evaporation losses. Samples *A* and *C* were filtered at 60° C. and samples *B*, *D*, and *E* were filtered at 90° C.

Table 4 shows a comparison of the characteristics of the filtered and unfiltered tars. In all cases the specific gravity was reduced by filtration. The kinematic viscosity of the filtered samples showed a slight decrease for samples *A* and *C*, and a considerable decrease for sample *B* which contained a high percentage of organic matter insoluble in carbon disulfide. For samples *D* and *E* the viscosity increased. This increase is believed to have been caused by evaporation losses. These samples (*D* and *E*) were more difficult to filter than the coal tars.

The tar-insoluble material shown in the last column of table 4 represents the carbonaceous material that is suspended in the tar and can be seen under the microscope. The percentage of this material was calculated from the difference between the solubilities in carbon disulfide of the unfiltered and filtered tars. It is not the difference between the values shown in table 4 since the percentage for the filtered tar is not on the basis of the total tar and this correction must be considered. The percentage of tar-insoluble material showed wide variations among the different types of tars. It is of interest to note that even though the organic material insoluble in carbon disulfide for the unfiltered coal tars *A* and *C* and the unfiltered water-gas tars *D* and *E* showed only minor differences, there is a large difference in the amount of suspended matter present. All of the filtered tars were essentially clear of any suspended particles when viewed under the microscope.

As a further check on the efficiency of the filtration, determination was made of the amount of material in a 10-gram sample of the filtered horizontal retort tar *B*, that was insoluble in 100 milliliters of nitrobenzene. The result of this test was 0.02 percent insoluble organic material.

SEPARATION POINT RELATED TO BREAK-DOWN OF TAR

In order to study the phenomena accompanying the separation of the tar blend and to determine the effect of aging on the separation point, series of blends with Diesel oil were made for filtered and unfiltered portions of each base tar. The percentage of Diesel oil was varied from zero up to the separation point and observations made during a period of 30 days. The percentage of Diesel oil in the blend showing a slight separation for the various periods of aging are shown in table 5.

[2] *Physical properties of coal tars*, by E. W. Volkmann, E. O. Rhodes, and L. T. Work; Industrial and Engineering Chemistry, vol. 28, No. 6; June 1936; p. 721.

Table 3.—*Characteristics of bases and fluxes from known sources*

	Base tars					Fluxes			
						Coal-tar fluxes		Water-gas tar fluxes	
	Coke-oven tar	Horizontal-retort tar	Vertical-retort tar	Heavy water-gas tar	Heavy water-gas tar	Low boiling	Medium boiling	Low sulfonation	High sulfonation
Sample identification	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*
Grade	RT-4	RT-9	RT-4	RT-8	RT-11	Flux	Flux	Flux	Flux
Specific gravity at 25°/25° C	1.176	1.245	1.106	1.171	1.176	1.008	1.101	1.085	1.012
Engler specific viscosity:									
At 40° C	23.4		35.4			1.1	1.7	1.9	1.6
At 50° C			15.4						
Float test:									
At 32° C.....seconds		165.2		87.9	849.0				
At 50° C.....do		69.0		45.0	144.0				
Kinematic viscosity at 35° c. centistokes	322	29,900	429	10,900	306,000	2.0	11.0	13.4	9.2
Carbon disulfide solubility:									
Bitumen.....percent	94.20	76.63	94.41	92.51	93.28	99.95	99.54	99.91	99.90
Organic insoluble material...do	5.76	23.32	5.55	7.25	6.56	.05	.43	.08	.08
Inorganic insoluble material..do	.04	.05	.04	.24	.16	0	.03	.01	.02
Water content.....do	0	0	Trace	.5	0	0	Trace	0	1.4
Distillation (by weight):									
Total distillate:									
To 170° C.....percent	.29	0	.43	.47	.21	.52	.13	.45	1.11
To 200° C.....do	2.02	0	1.23	1.18	.83	5.21	.63	2.78	2.14
To 235° C.....do	6.79	1.74	5.52	4.23	4.28	66.94	2.23	13.59	11.49
To 270° C.....do	15.83	7.62	17.62	10.87	11.17	94.90	17.20	26.60	41.20
To 300° C.....do	23.39	12.42	27.12	19.10	19.45	97.57	37.11	37.22	57.55
To 355° C.....do	38.05	25.45	45.27	30.99	32.58	98.17	78.55	58.18	76.18
Residue from distillate to 300° C. percent	75.98	86.84	72.00	79.82	80.16	1.80	62.30	62.40	41.49
Softening point of residue above 300° C.....percent	38.0	52.2	39.6	62.8	60.9	(1)	(1)	29.6	29.4
Sulfonation index:									
On total distillate to 300° C	.3	.2	2.7	.3	.1	1.2	.3	.6	11.1
On distillate between 300–355° C	.1	.2	1.9	.1	.3	.1	.4	.1	1.9
On total distillate to 355° C	.4	.4	4.6	.4	.4	1.3	.7	.7	13.0
Separation point	27.5	22.5	20.0	32.5	25.0	(2)	50.0	55.0	27.5

[1] Too fluid for softening point determination.

[2] Miscible with Diesel oil in all proportions.

Table 4.—*Comparison of characteristics of filtered and unfiltered tars*

Sample identification	Specific gravity at 25° C.		Kinematic viscosity at 35° C.		Organic material insoluble in carbon disulfide		Tar-insoluble material [1]
	Unfiltered	Filtered	Unfiltered	Filtered	Unfiltered	Filtered	
			Centistokes	*Centistokes*	*Percent*	*Percent*	*Percent*
A	1.176	1.172	373	342	5.76	4.81	1.00
B	1.245	1.197	31,200	10,000	23.32	9.01	15.73
C	1.106	1.103	854	683	5.55	4.20	1.41
D	1.171	1.152	12,900	13,500	7.25	.85	6.45
E	1.176	1.161	378,000	415,000	6.56	1.87	4.78

[1] Calculated from differences in organic material insoluble in carbon disulfide for filtered and unfiltered tar. This represents the carbonaceous material suspended in the tar.

Table 5.—*Variation of separation point and precipitation point with age of blends tested*

Sample identification	Test method	Percentage of Diesel oil in blend showing slight separation after aging test blends for—				
		0	1 day	2 days	7 days	30 days
Sample *A*:						
Unfiltered	Separation point		27.5			22.5
Filtered	do	27.5	27.5	27.5	25.0	22.5
Do	Precipitation point [1]	22.5	20.0	20.0	20.0	20.0
Sample *B*:						
Unfiltered	Separation point	22.5	22.5			22.5
Filtered	do	27.5	22.5	22.5	22.5	22.5
Do	Precipitation point [1]	20.0	20.0	20.0	20.0	20.0
Sample *C*:						
Unfiltered	Separation point	20.0	20.0			20.0
Filtered	do	25.0	25.0	25.0	22.5	20.0
Do	Precipitation point [1]	22.5	20.0	20.0		17.5
Sample *D*:						
Unfiltered	Separation point	37.5	32.5			32.5
Filtered	do	37.5	37.5	37.5	35.0	32.5
Do	Precipitation point [1]	37.5	35.0	35.0	35.0	30.0
Sample *E*:						
Unfiltered	Separation point		25.0			25.0
Filtered	do	32.5	30.0			
Do	Precipitation point [1]	27.5	27.5			

[1] Determined by microscopic examination.

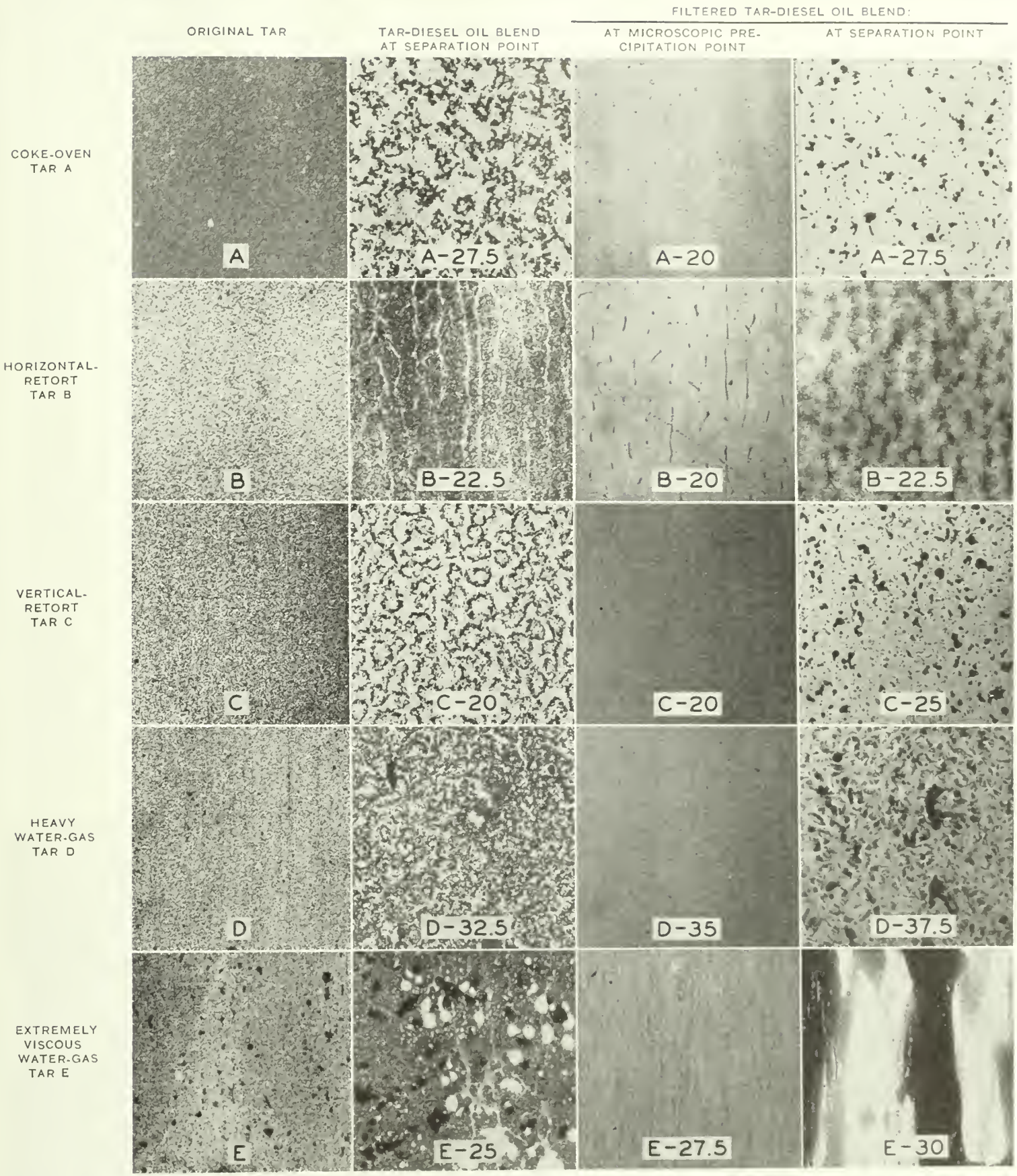

Figure 2.—Photomicrographs of blends of various tars with Diesel oil, showing (from left to right) the original tar, the tar–Diesel oil blend at separation point, the filtered tar–Diesel oil blend at microscopic separation point, and the filtered tar–Diesel oil blend at separation point. The type of tar and the percentage of Diesel oil in each blend are indicated at the bottom of each photograph by letter and number.

In addition to the usual observations for separation point, blends of the filtered tar were also examined under the microscope and the blend containing the lowest percentage of Diesel oil which showed the presence of precipitated particles or a separation into two phases, was recorded as the precipitation point. These data are also shown in table 5.

In all of the blends shown in table 5, the degree of the separation increased with the age of the blend. When separation first occurred the presence of two phases was often difficult to determine but upon standing there was a definite formation of a bottom phase, which became increasingly more viscous, and a top phase, which became less viscous and free of any flocculated particles. For all the unfiltered tars except sample *A*, there was no change in the separation point between 1 and 30 days. For sample *A*, the separation point decreased 5 percent in 30 days. For the filtered tars, the separation point decreased 5 percent from 1 to 30 days in all cases except for sample *B*. For sample *B*, the separation point was constant after 1 day, but there was a 5-percent decrease in the 1-day period. The observations with the microscope showed that there was only a slight change in the precipitation point after 1 day, and in the cases where a change occurred (sample *C*, 30 days, and sample *D*, 30 days) the amount of precipitation was extremely small. These results indicate that the changes in the appearance of the blends with age is caused chiefly by a mechanical settling of the flocculated particles and that the chemical equilibrium established after 1-day aging is only slightly affected by time.

The precipitation point determined with the microscope indicates a break-down in the structure of the tar and can be determined precisely. However, this determination requires the use of considerable special apparatus and is time consuming. The separation point is closely related to this precipitation point and the simplicity of the method makes it a more valuable laboratory test.

Microscopic study of all the blends involved in the determination of the separation points shown in table 5 showed that for the unfiltered tars there was a progressive flocculation of the dispersed phase with increasing percentages of Diesel oil. Photomicrographs were made to show this effect and also to show the appearance of the tars at the various critical points.

Photomicrographs of each base tar, the appearance of the blend at the separation point, the appearance of the filtered tar at the precipitation point, and the appearance of the filtered tar at the separation point are shown in figure 2. In each part of the figure, the type of tar and the percentage of Diesel oil in the blend are indicated by letter and number. The blends for all these photomicrographs were 1 day old and the pictures were taken of fresh slides that stood undisturbed for approximately 15 minutes, until all movement stopped. The slides were made with a sliding gage which gives a film thickness of 0.001 inch except those shown in figures 2*B* and the 2*E* series, which were 0.0005-inch films. No cover glass was used for any of these photomicrographs. The magnification in all cases is 58 diameters.

The coke-oven tar *A* has the most finely divided particles of any of the base materials studied. The separation point for this tar resulted from the flocculation of the dispersed phase and the precipitation of these conglomerates from the tar blend. When viewed immediately, the precipitated particles in the filtered tar had a fine lacy structure and were evenly distributed throughout the field. However, they quickly coalesced, forming closely clustered masses. The photomicrograph of the 1-day old blend shows these clusters after coalescence into larger groups (top right illustration, fig. 2).

The horizontal-retort tar *B* was very difficult to photograph because of its high concentration of free carbon. It was necessary to use a 0.0005-inch film thickness for the original material. The precipitated particles in the third illustration in the *B* series, figure 2, were evenly distributed when the slide was first made, but they quickly formed the pattern shown. The same is true for the fourth illustration in the *B* series. Small droplets of excess oil which had separated from the mass of the tar were visible under the microscope, but these are obscure in this photomicrograph.

The vertical-retort tar *C* (fig. 2*C*) showed essentially the same type of behavior as did the coke-oven tar *A*. One important difference between these two materials which cannot be shown by the photographs is the difference between the color and opacity of the continuous phase. The film for the coke-oven tar *A* is a brownish red and requires a considerably greater light intensity for photographing than does the yellow film for the vertical-retort tar *C*. The dispersed particles in the original tar are initially larger for sample *C* than for sample *A*.

The water-gas tar *D* (fig. 2*D*) shows evenly distributed small bodies throughout the tar with large, very black particles scattered unevenly. These particles will settle from the tar on long standing and are not an integral part of the tar system. For this material, the separation is chiefly a separation into two liquid phases. This is most clearly shown by the filtered tar blend in the last illustration in the *D* series, figure 2.

The extremely viscous water-gas tar *E* (fig. 2*E*) shows considerably more of the large carbon particles than does sample *D*. The separation in this case occurs when the tar cannot dissolve any more Diesel oil and the very light oils separate out, leaving the bulk of the tar with generally the same appearance as the original. Slides of this material are very difficult to make because of the lubri-

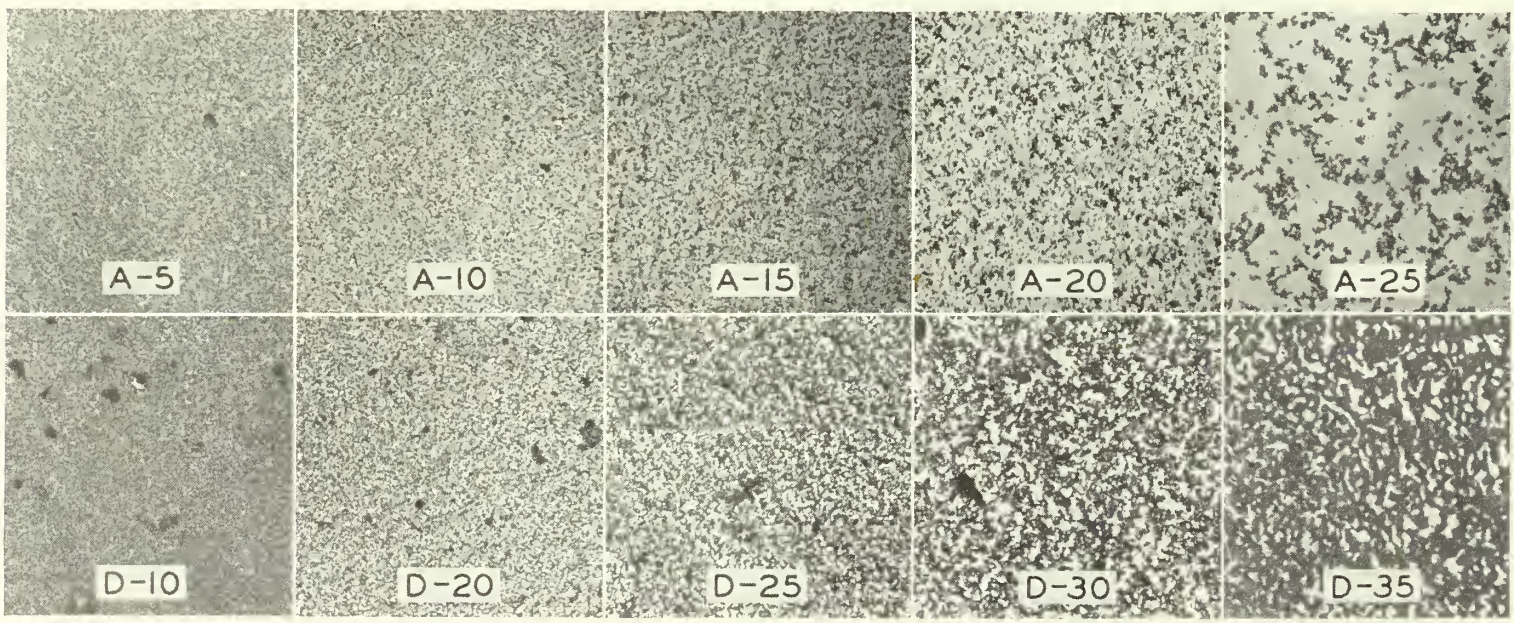

Figure 3.—Photomicrographs of blends of coke-oven tar A *and water-gas tar* D *with Diesel oil, showing progressive flocculation of the dispersed phase. The type of tar and the percentage of Diesel oil in each blend are indicated at the bottom of each photograph by letter and number.*

Table 6.—*Characteristics of special blends with heavy water-gas tar D*

Blend	Composition by volume			Specific gravity at 25° C.		Kinematic viscosity of unfiltered sample at 35° C.	Diesel oil added to reach separation point		Total Diesel oil in blend at separation point	
	Base tar *D*	Light water-gas tar *II*	Diesel oil	Unfiltered sample	Filtered sample		Unfiltered sample	Filtered sample	Unfiltered sample	Filtered sample
	Percent	*Percent*	*Percent*			*Centistokes*	*Percent*	*Percent*	*Percent*	*Percent*
a	65	35	0	1.144	1.136	452	37.5	37.5	37.5	37.5
b	65	28	7	1.129	1.126	353	32.5	32.5	37.2	37.2
c	65	21	14	1.111	1.104	250	27.5	27.5	37.6	37.6
d	65	14	21	1.095	1.086	191	17.5	17.5	34.8	34.8
e	65	7	28	1.075	1.071	196	10.0	10.0	35.2	35.2
f	65	0	35	1.061	1.054	252	0	0	35.0	35.0

cating effect of the separated oil which prevents an even distribution of the tar–Diesel oil film.

The photomicrographs (magnified 58 times) shown in figure 3 are typical examples of the progressive flocculation of the dispersed phase which occurs when increasing percentages of Diesel oil are added to the unfiltered tar. In these cases the 0.001-inch tar film was covered immediately with a cover glass which prevented the pattern formation previously discussed. The type of tar and the percentage of Diesel oil in the blends are indicated by letter and number, in each part of the figure. It is noted that the flocculation of the dispersed phase has progressed to an advanced stage before separation occurs. The blends shown in figures 3*A–25* and 3*D–30* are 2.5 percent below the separation point. The point at which visual separation occurs cannot be determined under the microscope by any sudden change in the appearance of the blends of the unfiltered tar.

VISCOSITY OF TAR HAS SLIGHT EFFECT ON SEPARATION POINT

In these series of blends there were wide differences in the viscosities of the original materials, and the various amounts of Diesel oil added gave blends that were widely different in viscosity at the separation point. A limited study of blends of approximately the same viscosity was made, using the heavy water-gas tar *D* as the base material. Six blends were made, with a constant volume of 65 percent of the base material and 35 percent of a flux made up of combinations of the light water-gas tar *II* and Diesel oil. The proportion of Diesel oil in the flux varied from 0 to 100 percent in 20 percent increments. The composition and characteristics of these blends are shown in table 6. The appearance of each of these materials 1 day after blending is shown in figure 4, and the same identifying letters are used as in table 6. These photomicrographs, of 0.001-inch films magnified 58 times, show progressive flocculation with increasing percentages of Diesel oil in essentially the same manner as did the base materials already shown.

The last two columns of table 6 show only a slight downward trend in the total amount of Diesel oil at the separation point as the amount of true flux, *II*, is decreased. If the viscosity of the combination of the base tar and flux *II* be considered, it is seen that this series is equivalent to a series of tars ranging from approximately RT–4 (viscosity range of blend *a*) to RT–8 (viscosity range of base tar) and thus it is indicated that the viscosity has only a minor influence on the separation point. The kinematic viscosity data shown in table 6 indicate a break in the viscosity–composition curve of the unfiltered tars caused by the increasing flocculation. The viscosities of blends *e* and *f* rise slightly instead of decreasing as would be expected by the trend of decreasing viscosity for additional Diesel oil established by the blends *a* through *d*.

UNSULFONATABLE HYDROCARBONS AT SEPARATION POINT VARY

In order to study the effect of unsulfonatable hydrocarbons on the properties of RT–2, RT–3, and other grades of road tar, a number of combinations of the base and flux materials shown in table 3 were made. Diesel oil was also added when materials of very high sulfonation indexes were desired. To obtain base materials of the same order of viscosity, it was necessary to distill samples *A* and *C* to 270° C. The residues from these distillations were then used as the base material in all the blends for samples *A* and *C*.

Table 7 shows the results of various tests made on a series of blends for each base tar. The blends with suffix number *1* represent a balanced road material made up in accordance with the suggestions of the producer who supplied the material. These blends are believed to be representative of the best practices for manufacturing road tars of the RT–2 and RT–3 grades using only a single type of base material. Photomicrographs of these blends show them to have the same general appearance and very nearly the same degree of dispersion as do the original base tars. The blends with suffix number *2* contain the same percentages by weight of base and light flux, but the medium fluxing material has been replaced with the same percentage of the high sulfonation-index light water-gas tar, *I*. In all cases, except for sample *A* in which no change was noted, the separation point was less when sample *I* was used. No appreciable flocculation could be detected in these blends.

The blends with suffix numbers *3* and *4* were made using only the light water-gas tars as fluxes. For the number *3* blends sample *I* (high sulfonation index) was used, and for the number *4* blends sample *II* (low sulfonation index) was used. In these cases the separation point was less for the number *3* blends, which contained the greater amount of unsulfonatable hydrocarbons, than for the number *4* blends. The blends with suffix number *5*, made with samples *A* and *D*, each contain the same percentage of unsulfonatable hydrocarbons as did the number *3* blends of the respective material, but the source of the flux and the sulfonation residue differs. The flux for the number *5* blends was a mixture of Diesel oil and sample *II*. In each case the separation point was 2.5 percent (one incre-

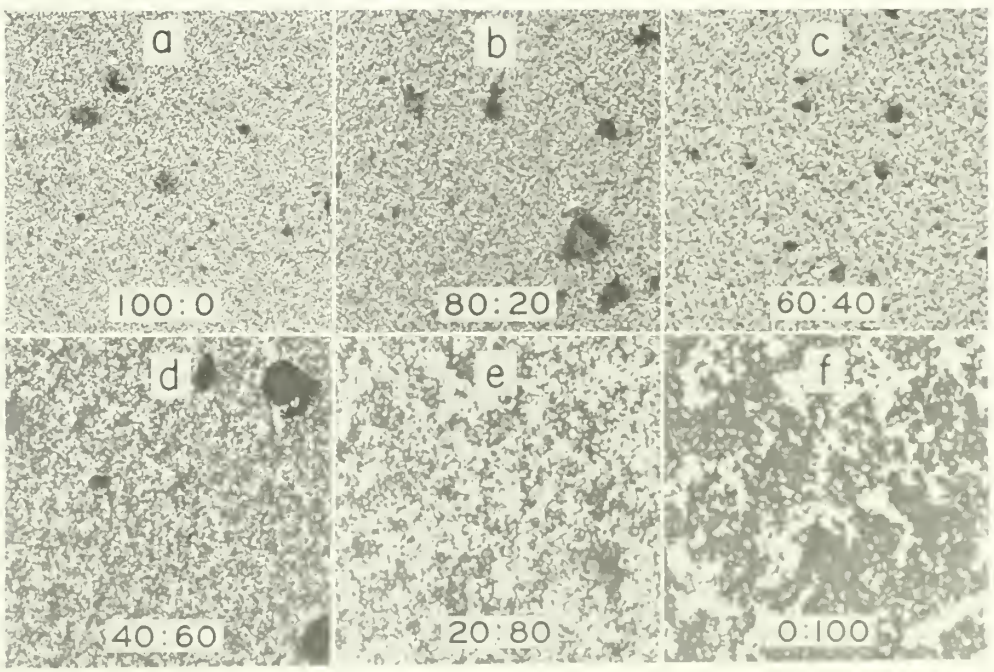

Figure 4.—Photomicrographs of blends of water-gas tar D containing 65 percent by volume of the base tar and a flux of various proportions of light water-gas tar II and Diesel oil. The proportions of the flux in each blend are indicated at the bottom of each photograph, the first number being the percentage of tar II, and the second number the percentage of Diesel oil.

Table 7.—*Characteristics of blends of base tars with various fluxes*

Blend identification	Composition									Specific gravity at 25°/25° C.	Kinematic viscosity at 35° C.	Sulfonation index to 355° C. (calculated)	Separation point	Total volume of unsulfonatable distillate at separation point
	Base			Medium flux			Light flux							
	Identification	Weight	Volume	Identification	Weight	Volume	Identification	Weight	Volume					
		Percent	*Percent*		*Percent*	*Percent*		*Percent*	*Percent*		*Centistokes*			*Percent*
Blends with tar *A:*														
A-1	*A* [1]	70.0	67.7	*H*	15.0	16.0	*F*	15.0	17.3	1.164	298	0.4	27.5	17.9
A-2	*A* [1]	70.0	65.8	*I*	15.0	17.0	*F*	15.0	17.1	1.150		2.3	27.5	19.5
A-3	*A* [1]	70.0	66.0	*I*	30.0	34.0				1.149	847	5.0	20.0	17.4
A-4	*A* [1]	70.0	67.4	*H*	30.0	32.6				1.179		.3	27.5	17.9
A-5	*A* [1]	70.0	65.2	*H*	24.9	27.9	Diesel oil	5.1	6.9	1.135	835	5.0	22.5	19.0
A-6	*A* [1]	70.0	63.8	*I*	15.0	16.4	do	15.0	19.8	1.111	851	13.0	2.5	15.8
A-7	*A* [1]	70.0	62.7	*I*	7.5	8.1	do	22.5	29.2	1.093		17.6	[2]	
Blends with tar *B:*														
B-1	*B*	70.0	66.3	*G*	15.0	16.1	*F*	15.0	17.6	1.179	232	.4	12.5	8.4
B-3	*B*	70.0	65.5	*I*	30.0	34.5				1.160	1,158	4.2	5.0	7.9
B-6	*B*	70.0	63.3	*I*	15.0	16.7	Diesel oil	15.0	20.0	1.126		13.1	[2]	
Blends with tar *C:*														
C-1	*C* [1]	68.0	66.1	*G*	16.0	16.2	*F*	16.0	17.7	1.116	264	2.2	32.5	22.5
C-2	*C* [1]	68.0	65.1	*I*	20.0	21.8	*F*	12.0	13.1	1.100		4.7	27.5	21.4
C-3	*C* [1]	68.0	65.2	*I*	32.0	34.8				1.101	885	6.1	17.5	16.7
C-4	*C* [1]	68.0	66.7	*H*	32.0	33.3				1.127		2.1	30.0	20.8
C-6	*C* [1]	68.0	65.5	*I*	16.0	15.7	Diesel oil	16.0	18.8	.991	863	14.0	[2]	
C-7	*C* [1]	68.0	64.5	*I*	8.9	7.7	do	24.0	27.8	.976		18.7	[2]	
Blends with tar *D:*														
D-1	*D*	67.0	64.8	*G*	20.0	20.6	*F*	13.0	14.6	1.132	173	.5	40.0	25.9
D-2	*D*	67.0	63.6	*I*	20.0	22.0	*F*	13.0	14.4	1.112		3.0	37.5	26.1
D-3	*D*	67.0	63.7	*I*	33.0	36.3				1.109	452	4.5	27.5	21.2
D-4	*D*	67.0	65.3	*H*	33.0	34.7				1.141		.4	40.0	25.9
D-5	*D*	67.0	64.1	*H*	27.5	28.4	Diesel oil	5.6	7.5	1.121	452	4.5	30.0	22.8
D-6	*D*	67.0	61.4	*I*	16.5	17.5	do	16.5	21.1	1.074	298	14.4	10.0	20.1
D-7	*D*	67.0	60.8	*I*	8.3	8.7	do	24.7	30.9	1.056	347	19.2	2.5	20.4
D-8	*D*	67.0	60.0	*I*	5.0	5.1	do	28.0	34.9	1.048		21.2	[2]	
Blends with tar *E:*														
E-1	*E*	62.5	60.2	*G*	25.0	25.7	*F*	12.5	14.1	1.133	263	.6	35.0	22.8
E-2	*E*	62.5	58.9	*I*	25.0	27.4	*F*	12.5	13.7	1.108		3.7	32.5	23.6
E-3	*E*	62.5	59.9	*I*	37.5	41.1				1.105	1,010	5.1	22.5	18.7
E-4	*E*	62.5	60.6	*H*	37.5	39.4				1.140		.5	35.0	22.8
E-6	*E*	62.5	56.6	*I*	18.75	19.7	Diesel oil	18.75	23.7	1.066	804	16.3	2.5	18.5
E-7	*E*	62.5	55.4	*I*	9.4	9.7	do	28.1	34.9	1.044		21.8	[2]	

[1] Residue from distillation to 270° C.

[2] Blend separated.

ment) higher for blend *5* than for blend *3*. Photomicrographs showed no discernible difference in the degree of dispersion in these tars.

The blends with suffix numbers *6*, *7*, and *8*, together with number *3*, form a series in which, for each tar, a constant percentage by weight of base tar was blended with a flux in which the amount of unsulfonatable hydrocarbons varied greatly. The flux for the number *6* blends contained 50 percent by weight of Diesel oil and 50 percent by weight of sample *I*. The flux for the number *7* blends contained 75 percent (by weight) Diesel oil and 25 percent sample *I*. For the number *8* blend with sample *D*, the flux was 85 percent (by weight) Diesel oil and 15 percent sample *I*. The blends for each of the base materials showed the same behavior as the series reported in table 6 for the water-gas tar *D*; that is, those blends with smaller amounts of tar flux and greater amounts of Diesel oil showed increasing flocculation of the dispersed phase as the percentage of Diesel oil increased.

Unusual behavior was shown by the blends from sample *B*, the horizontal-retort tar. These samples formed a gel with Diesel oil at percentages considerably lower than the point at which two distinct phases could be detected. Microscopic examination showed that this gelling was caused primarily by the high concentration of the flocculated particles. These formed a lacy network which, upon standing, developed a structural strength from the coherence of the tar particles. After standing overnight, containers holding these samples could be inverted without causing flow. After stirring, however, the material flowed freely. Special blends were made using the filtered tar as base, with the same percentages of flux as was used in the *B–1* and *B–3* blends. This filtered *B–1* blend had a precipitation point (microscopic) of 12.5 percent, and the filtered *B–3* blend had a precipitation point of 2.5 percent. These points were in close agreement with the point of gelling, 12.5 percent for *B–1* and 5.0 percent for *B–3*, determined for the unfiltered blends. The separation point for the filtered blends was 20 percent for *B–1* and 10 percent for *B–3*.

The final column in table 7 shows the total percentage of unsulfonatable material present in the test blend at the separation point, as calculated from the amount in the Diesel oil added and from that contained in the tar as determined by the sulfonation index. The amount of unsulfonatable hydrocarbons that can be tolerated by each tar is approximately constant. Much of the variation in each group of blends is caused by the method of test for the separation point, since the Diesel oil was added in increments of 2.5 percent, each increment representing 1.6 percent of unsulfonatable material in the blend.

Another source of variation is the large difference in the amount of good tar flux present in the blends, since this material will tolerate more Diesel oil than will the base tar. The average value of the percentage of paraffinic and naphthenic hydrocarbons required to cause separation varied for each base material, with the lower values being obtained with the coal tars. These values were 17.9 percent for sample *A*, 8.1 percent for sample *B*, and 20.3 percent for sample *C*. The values for the water-gas tars were 23.2 percent for *D*, and 21.3 percent for *E*.

The blends reported in table 7 were made with a limited number of materials and in most cases essentially all of the unsulfonatable hydrocarbons present in the various blends were from the Diesel-oil flux. In order to obtain data that would be more representative of the materials which are encountered in practice, a number of trade tars were tested. These tars were selected so as to give as wide a range in the sulfonation index as was possible. The very high sulfonation-index tars selected include almost all such tars submitted to the laboratory over a period of several years and are not typical of the usual road tar submitted for routine analysis. Some of the test characteristics for these materials are shown in table 8. Comparison of the separation point with the sulfonation index shows that there is a definite trend toward a decreasing separation point for an increasing sulfonation index of the total distillate to 355° C. This is also shown by the close agreement of the total amount of unsulfonatable distillate required to cause separation (last column of table 8), since for an exact inverse relation between sulfonation index and separation point this value would be a constant.

There are some materials that show deviations in this value which are considerably larger than can be attributed to experimental error. This is probably caused by a low resistance to flocculation of the particular base tar or by an unbalanced blend of tar materials. In general, however, neither the grade of the material nor its source (as represented by the producer) made any definite difference in the amount of unsulfonatable hydrocarbons that could be tolerated by the tar. The average percentage of unsulfonata-

ble hydrocarbons in the blend at the separation point is 21.9. The average deviation from this mean is ± 2.2 percent. Since, as previously noted, the procedure for the determination of the separation point requires the Diesel oil to be added in increments of 2.5 percent, which equals 1.6 percent unsulfonatable material, it is concluded that most tar being used for the manufacture of road materials will separate with essentially the same percentage of paraffinic and naphthenic hydrocarbons.

Microscopic examinations of the trade tars show that, as was the case with the laboratory blends, the degree of flocculation of the particles in the tar increases as the separation point decreases. This is shown by the photomicrographs in figure 5, in which *1* and *12* are samples so identified in table 8 (sample *X* illustrated in figure 5 is not included in table 8). It should also be noted that those materials that have lower separation points than would be indicated by the sulfonation indexes are flocculated to a greater degree than are those having separation points more consistent with the sulfonation indexes. This is indicated by the appearance of the tars shown in figures 5–*12* and 5–*X*. There is a wide difference in the separation points of the two tars—12.5 for the sample in figure 5–*12* and 2.5 for the sample in figure 5–*X*—while the sulfonation indexes of the total distillate to 355°C. are 7.5 and 8.8, respectively, which is equivalent to a difference of only 1.3 percent in unsulfonatable material.

KINEMATIC VISCOSITY—COMPOSITION CURVES

The effect of the unsulfonatable hydrocarbons on the kinematic viscosity of tars was studied by blending the base tars and selected trade tars with various percentages of either Diesel oil or a coal-tar distillate (sample *F*, table 3). The kinematic viscosities of these blends at 35° C. were determined by means of a capillary viscosimeter.[3]

In order to insure uniform blending, the blends of the base tars were prepared by adding the desired amount of flux and warming in a closed container on the steam bath to approximately 60° C. The sample was then stirred until a uniform blend was obtained. The blends with the RT–2 and RT–3 materials were made at room temperature.

The viscosity tests of all the blends in which the coal-tar distillate was the diluent were not affected by the period of aging or the amount of stirring of the material before testing. It was concluded that in the proportions used for these tests the tar distillate was completely compatible with all the tars.

The blends with low percentages of Diesel oil in which the flocculation was not excessive generally had the same viscosity at the top and bottom of the sample and also after standing overnight without stirring. The blends with percentages of Diesel oil near or greater than the separation point gave erratic results. Such blends were allowed to stand overnight in the viscosimeter tube and brought to test temperature without stirring. The capillary tube was then immersed to a depth of 0.2 centimeters for the first test and to a depth of 2.0 centimeters for the second test. In cases where a definite separation into two phases had occurred, the depth of immersion of the capillary tube for the second test was adjusted so that the tip of the tube was in the lower phase. The viscosity of each phase was thus

Table 8.—*Test characteristics for various grades of road tars*

Identification	Producer	Consistency: Engler specific viscosity at 40° C.	Consistency: Float test at 32° C.	Specific gravity at 25°/25° C.	Softening point of residue above 300° C.	Sulfonation index: Distillate to 300° C.	Sulfonation index: Distillate from 300° C. to 355° C.	Sulfonation index: Total distillate to 355° C.	Separation point	Volume of unsulfonatable distillate at separation point
			Seconds		° C.					*Percent*
Grade RT–2 tar, sample:										
1	Z	12.2		1.152	50.7	0.3	0.1	0.4	32.5	21.3
2	Z	12.9		1.148		.9	.1	1.0	27.5	18.7
3	X	10.1		1.128	49.6	1.5	.7	2.2	32.5	23.3
4	Y	12.3		1.122		1.8	.7	2.5	32.5	23.6
5	X	11.8		1.108	29.2	2.2	2.0	4.2	25.0	20.7
6	Z	11.3		1.137	33.0	2.9	1.4	4.3	30.0	24.1
7	X	11.7		1.115	46.8	3.3	1.2	4.5	27.5	22.6
8	X	9.1		1.126	40.4	3.2	1.6	4.8	27.5	23.0
9	X	11.8		1.102	47.2	4.0	1.4	5.4	27.5	23.6
10	W	12.7		1.153	42.8	5.2	.5	5.7	12.5	14.6
11	X	11.2		1.111	59.2	6.4	1.0	7.4	27.5	25.8
12	X	12.1		1.098	35.7	5.4	2.1	7.5	12.5	16.2
13	Y	11.5		1.096		10.3	.7	11.0	17.5	23.3
14	X	12.9		1.084	33.5	6.9	4.5	11.4	15.0	22.0
Average for grade										21.6
Grade RT–3 tar, sample:										
15	U	17.3		1.151	50.8	.6	.2	.8	37.5	24.9
16	Z	16.2		1.138	37.0	.8	.3	1.1	37.5	25.3
17	U	21.2		1.127	57.1	2.2	.8	3.0	35.0	25.8
18	Y	20.5		1.122	50.4	2.2	1.0	3.2	27.5	21.2
19	Y	18.2		1.146	28.2	2.8	.8	3.6	27.5	21.7
20	Y	17.0		1.136	33.0	3.9	1.0	4.9	25.0	21.6
21	Y	15.9		1.130	36.5	4.9	1.5	6.4	25.0	23.2
22	W	15.0		1.155	42.9	6.0	.4	6.4	17.5	18.6
23	Z	13.3		1.128		7.6	.1	7.7	22.5	23.1
24	X	20.0		1.104	40.7	7.2	2.6	9.8	17.5	22.0
25	X	13.8		1.087	42.8	7.7	3.4	11.1	7.5	16.9
26	X	14.9		1.075	44.5	10.2	3.1	13.3	12.5	22.3
27	X	13.3		1.090	46.5	10.8	3.0	13.8	20.0	27.8
28	X	13.8		1.085	42.0	12.4	2.4	14.8	7.5	20.9
29	X	20.7		1.055	48.2	17.0	2.2	19.2	2.5	21.9
Average for grade										22.5
Grade RT–4 tar, sample:										
30	U	32.5		1.135	60.4	2.2	.5	2.7	37.5	27.1
31	X	23.1		1.115	47.5	3.4	1.4	4.8	27.5	23.0
32	X	25.6		1.108	32.6	7.1	4.1	11.2	10.0	18.8
33	X	28.4		1.086	56.5	16.0	2.4	18.4	5.0	23.2
Average for grade										23.0
Grade RT–5 tar, sample:										
34	Z	[1]25.0		1.180	55.4	.2	.1	.3	25.0	16.3
35	T	[1]22.2		1.159	39.6	.5	.2	.7	32.5	21.5
36	U	[1]22.6		1.140	59.4	1.7	.6	2.3	32.5	23.3
Average for grade										20.4
Grade RT–6 tar, sample:										
37	U		45	1.232	58.5	.3	.2	.5	27.5	18.2
38	U	[1]31.6		1.145	60.0	1.5	.8	2.3	32.5	23.3
Average for grade										20.7
Grade RT–8 tar, sample:										
39	U		117	1.196	73.3	.1	.1	.2	30.0	19.4
40	X		85	1.212	42.6	.4	.2	.6	27.5	18.3
41	S		102	1.166	41.6	.9	1.0	1.9	30.0	21.5
42	X		106	1.182	49.2	4.5	1.2	5.7	25.0	22.8
43	X		94	1.154	60.8	7.1	1.6	8.7	20.0	24.2
Average for grade										21.2
Average for all grades										21.9

[1] Specific viscosity at 50° C.

[3] This viscosimeter and the procedure for making the test is the same as that described in the report *The determination of the kinematic viscosity of petroleum asphalts with a capillary viscosimeter*, by R. H. Lewis and W. J. Halstead; PUBLIC ROADS, vol. 21, No. 7; September 1940; p. 127.

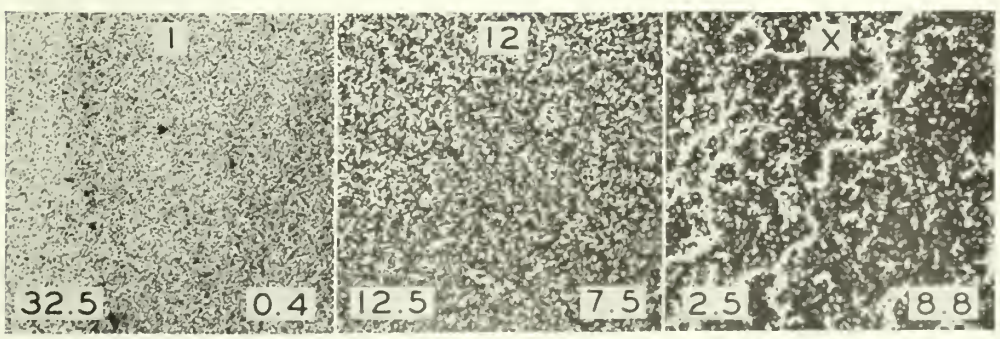

Figure 5.—Photomicrographs of trade tars showing variations in degree of flocculation with variations in the separation point. The percentage of Diesel oil added to the tar at the separation point is indicated by the left-hand number at the bottom of each photograph; the sulfonation index on the total distillate to 355° C. by the right-hand number.

Table 9.—*Kinematic viscosity of base tars blended with various percentages of Diesel oil and coal-tar distillate*

Identification of tar and volume of diluent in sample	Kinematic viscosity at 35° C.		
	Coal-tar distillate blends	Diesel-oil blends	
		Top	Bottom
Coke-oven tar *A* [1] with diluent percentage of:	*Centistokes*	*Centistokes*	*Centistokes*
0	120,000	120,000	120,000
10.0	4,800	8,300	8,300
20.0	609	1,700	1,700
22.5		1,530	1,530
25.0		1,470	1,470
27.5		1,490	1,830
30.0	127	840	3,080
40.0	41	5	7,130
Horizontal-retort tar *B* with diluent percentage of:			
0	29,000	29,000	29,000
10.0		4,930	4,930
12.5		3,720	3,720
15.0		2,980	2,980
17.5		3,120	3,120
20.0	755	2,580	2,580
22.5		2,820	2,820
25.0	236	400	
30.0	140		2,960
40.0	62		12,450
Vertical-retort tar *C* [1] with diluent percentage of:			
0	68,800	68,800	68,800
10.0	5,100	10,700	10,700
20.0	289	3,150	3,150
22.5		2,650	2,650
25.0		1,850	1,850
27.5		66	2,430
30.0	142	51	4,100
40.0	50	16.5	105,000
Heavy water-gas tar *D* with diluent percentage of:			
0	10,900	10,900	10,900
10.0		2,450	2,450
17.5	410		
20.0		960	960
22.5	219		
30.0	87		
32.5		270	270
35.0		200	390
40.0		13.5	37,000
45.0		5.8	17,000
Heavy water-gas tar *E* (extreme case) with diluent percentage of:			
0	306,000	306,000	306,000
10.0	17,800	43,700	43,700
20.0	1,980	15,000	15,000
22.5		3,270	3,270
25.0		4,050	15,000
27.5		2,560	
30.0	320	16.6	28,500
40.0	76		1,050,000

[1] Residue from distillation to 270° C.

determined. Because of the grainy or flocculated characteristics of the lower phase, check results varied greatly and only an approximate average value could be determined for this portion of the material.

Tables 9, 10, and 11 show the results of these viscosity tests. Table 9 shows the data for the base tars, table 10 gives the results for the RT–2 and RT–3 tars made by fluxing the base tars in the laboratory, and in table 11 are shown representative samples of RT–2 and RT–3 trade tars.

These data were plotted, using a viscosity scale the same as that for the modified viscosity temperature charts (A. S. T. M. method D 341–39) for the ordinates. A natural scale for the percentage of Diesel oil in the blend was used for the abscissae. All the curves, with the exception of that for the RT–3 tar made with horizontal-retort tar *B* as the base, were similar in appearance.

Typical examples of these curves are shown in figure 6 for the blends of sample *A* (table 9), the coke-oven base tar (270° C. distillation residue); for sample *D* (table 9), made with the heavy water-gas tar; for sample *1* (table 11), the RT–2 trade tar; for sample *A–1* (table 10), the RT–3 tar made from the coke-oven tar base; and for sample *25* (table 11), the RT–3 trade tar. The peculiar behavior of tar–Diesel oil blends of sample *B–1* (table 10), the RT–3 from the horizontal-retort tar, is also shown in figure 6. On the same grid is also shown a curve for coal-tar distillate blends. The data for this curve are as follows:

Volume of diluent in sample	*Kinematic viscosity at 35° C.*
Percent	*Centistokes*
0	232
10	89.5
20	47.3
30	31.1

In figure 6, the coke-oven tar *A* shows the effect on the viscosity when the separation is that of excess "oil" separating from the tar. In this case there is a considerable range in which there is only a very small change in the viscosity, with increasing amounts of Diesel oil. After the separation point has been exceeded the viscosity of the top phase becomes very low while the bottom shows a more gradual increase. For sample *A–1*, an RT–3 from coke-oven tar, there is no wide range of equal viscosity. Upon separation both phases retain liquid characteristics and the difference between the value for the viscosity of each phase increases more slowly. The separation for base tar *D* may also be classed as a separation into two liquid phases and the curve has the same general shape as that for the RT–3 (sample *A–1*). For the RT–2 trade tar (sample *1*) and for the RT–3 trade tar (sample *25*), the separation is a settlement of flocculated particles with the formation of a grainy mass after standing. The RT–2 is a low sulfonation residue tar with a separation point of 32.5 percent. The RT–3 is a high sulfonation tar and has a separation point of 7.5 percent. Both curves show a very rapid increase in the viscosity of the bottom phase.

For all the blends made with coal-tar distillate, the curves are approximately straight lines, with greater slopes than for the initial portion of the curves for the Diesel-oil blends for the same tar.

The peculiar behavior of sample *B–1*, the RT–3 made from the horizontal-retort tar, results from the gelling action caused by the high concentration of the flocculated particles. As the amount of Diesel oil is increased, the size and concentration of the flocculated particles increases and thus the resistance to flow also increases and the apparent viscosity of the blend increases. When the amount of Diesel oil increases to such an extent that an excess of oil is present this separates out on top while the bottom retains much the same appearance as the gel except that it becomes increasingly more viscous.

In general, the erratic changes in curvature of these viscosity–composition curves with increasing amounts of Diesel oil reflect the developments of incompatability of the components of the blend. This was shown by the studies with the microscope to result in the flocculation of the dispersed phase followed by the precipitation of some of the constituents of the continuous phase. The percentage of Diesel oil in the blend at the point of separation for the viscosity curves for the top and bottom of the samples corresponds closely to the separation point determined in the usual manner. Comparisons of these values are shown in table 12. Exact agreement is shown in five cases and in eight cases the separation point determined by the usual procedure was 2.5 percent (one increment) higher than that determined from the viscosity curves. In one case (sample *C*) the separation point by the usual procedure was 5 percent lower than that from the curves.

UNIFORMITY CHECKED BY SEPARATION-POINT TEST

While the separation point is not an exact measurement of a distinct property of a tar, these results show it to be a valuable indication of the compatibility of the components in any tar blend. This compatibility is closely related to the amount of unsulfonatable hydrocarbons present in the tar, but the relation between the separation point and the sulfonation index is not sufficiently exact to enable the separation-point test to be used in lieu of the sulfonation-index test. Further study of road tars with this test or similar tests based on the same principle to develop correlations with service behavior is believed necessary before it can be used in specifications.

The simplicity of the test suggests several practical uses by those engaged in testing or conducting research on road tars. One such

application is its use as a control test to check the uniformity of shipments from the same source and to indicate contamination with petroleum products if this should occur. This is illustrated by the results obtained on two samples of the same RT-2 grade road tar. This material was tested and the storage tank sealed. The sulfonation index was 4.5 (on the total distillate to 355° C.) and the separation point was 27.5 percent. The first distributor load from the storage tank was tested and the separation point was found to be 5 percent. The sulfonation index was 18.0 (on the total distillate to 355° C.). Investigation showed that the distributor had been filled with cut-back asphalt, RC-2, the previous day and apparently had not been cleaned before loading with tar. While overnight settling is necessary for a final result, definite indications of unusual conditions are obtained in the laboratory in only a few minutes by the use of the separation-point test.

Another application of this test is its use in research investigations as an indication of the compatibility of the constituents and as a measure of the changes in compatibility of the various tars being studied. In such applications, exact duplication of the procedure and flocculating agent (Diesel oil) used in this work would not be necessary as long as the same material and procedure is used throughout a series of tests.

THEORETICAL EXPLANATION OF DIESEL-OIL ACTION

A theoretical explanation of the phenomena shown in the test for separation point is suggested by the correlation of the results with the theory of the structure of tar and other bituminous materials. Most authors attribute changes in the dispersion characteristics or mutual solubility of the hydrocarbons to a change in surface and interfacial tension as a result of some outside influence. Oliensis discussed this theory in explaining the fundamental significance of the Oliensis spot test for asphalts.[4]

He described the action of the naphtha used in the test as weakening the surface tension of the lighter phases in the asphalt, thereby increasing the interfacial tension between them and the heavier phases, and impairing the mutual solubility. The degree of impairment is made a constant in the Oliensis test. The asphalts that develop negative spots have mutual solubilities (phase relations) which are so stable they are not destroyed by the standard amount of naphtha; hence no insoluble dispersion results. On the other hand, those materials giving positive spots do not have stabilities sufficient to prevent incompatibility of certain constituents when subjected to the action of the solvent, and a precipitation of these constituents results. The separation-point test for tars involves the same fundamental principles. In this case the "disturbing influence" or "flocculating force" is varied by adding different amounts of Diesel oil. The value of this flocculating force necessary to produce the same condition in all tars is measured, and is represented by the amount of Diesel oil required to cause separation into two phases. This has been previously shown to be very closely related to the point at which some constituents that were originally

[4] *Fundamental significance of Oliensis spot test—Quantitative tests for homogeneity*, by G. L. Oliensis; Proceedings of the 44th annual meeting of the American Society for Testing Materials, vol. 41; 1941; p. 1108.

Table 10.—*Kinematic viscosity of fluxed tars blended with various percentages of Diesel oil*

Identification of tar and volume of Diesel oil in sample	Kinematic viscosity at 35° C.	
	Top	Bottom
Sample *A-1* (RT-3 tar) with diluent percentage of:	*Centistokes*	*Centistokes*
0	191	191
10.0	101	104
20.0	75	75
25.0	56	56
27.5	51	51
30.0		74
32.5	20	
37.5	10	125
Sample *B-1* (RT-3 tar) with diluent percentage of:		
0	232	232
5.0	168	168
10.0	153	153
15.0	304	304
17.5	122	422
20.0	501	501
22.5	600	600
25.0	300	
27.5		3,800
30.0	22.6	24,140
35.0	9.1	(1)
45.0	3.2	(1)
Sample *C-1* (RT-3 tar) with diluent percentage of:		
0	186	186
15.0	68	68
27.5	30	30
30.0	24	24
35.0	19	19
40.0	12	(1)
45.0	7	(1)
Sample *D-1* (RT-2 tar) with diluent percentage of:		
0	109	109
10.0	70	70
20.0	53	53
35.0	18	18
40.0	6.3	330
45.0	6.4	138
50.0	3.0	(1)
Sample *E-1* (RT-3 tar) with diluent percentage of:		
0	234	234
10.0	124	124
20.0	73	73
30.0	61	61
32.5	82	82
35.0	22	87
37.5	14	(1)
40.0	9.1	(1)

[1] Material semisolid, wide variations in results makes actual viscosity indeterminable.

Table 11.—*Kinematic viscosity of trade tars blended with various percentages of Diesel oil and coal-tar distillate*

Identification of tar and volume of diluent in sample	Kinematic viscosity at 35° C.		
	Coal-tar distillate blends	Diesel-oil blends	
		Top	Bottom
Sample *I* (RT-2 tar) with diluent percentage of:	*Centistokes*	*Centistokes*	*Centistokes*
0	170	170	170
10.0	69	95	95
20.0		54	54
25.0	23.2		
30.0		43	43
32.5		28	72
35.0		17.1	7,970
37.5		11.7	3,820
40.0	9.8	4.6	
45.0		3.3	
Sample *8* (RT-2 tar) with diluent percentage of:			
0	88	88	88
10.0	41.3	61	61
20.0		36	36
22.5		27	27
25.0	15.6	25	25
27.5		15.1	422
30.0		18.7	169
32.5		11.6	86
35.0		11.6	
40.0	8.6		
Sample *10* (RT-2 tar) with diluent percentage of:			
0		227	227
5.0		187	187
10.0		233	233
12.5		231	231
15.0		110	431
20.0		17	1,930
Sample *25* (RT-3 tar) with diluent percentage of:			
0		266	266
2.5		260	260
5.0		201	201
7.5		175	175
10.0		119	233
15.0		10.7	187

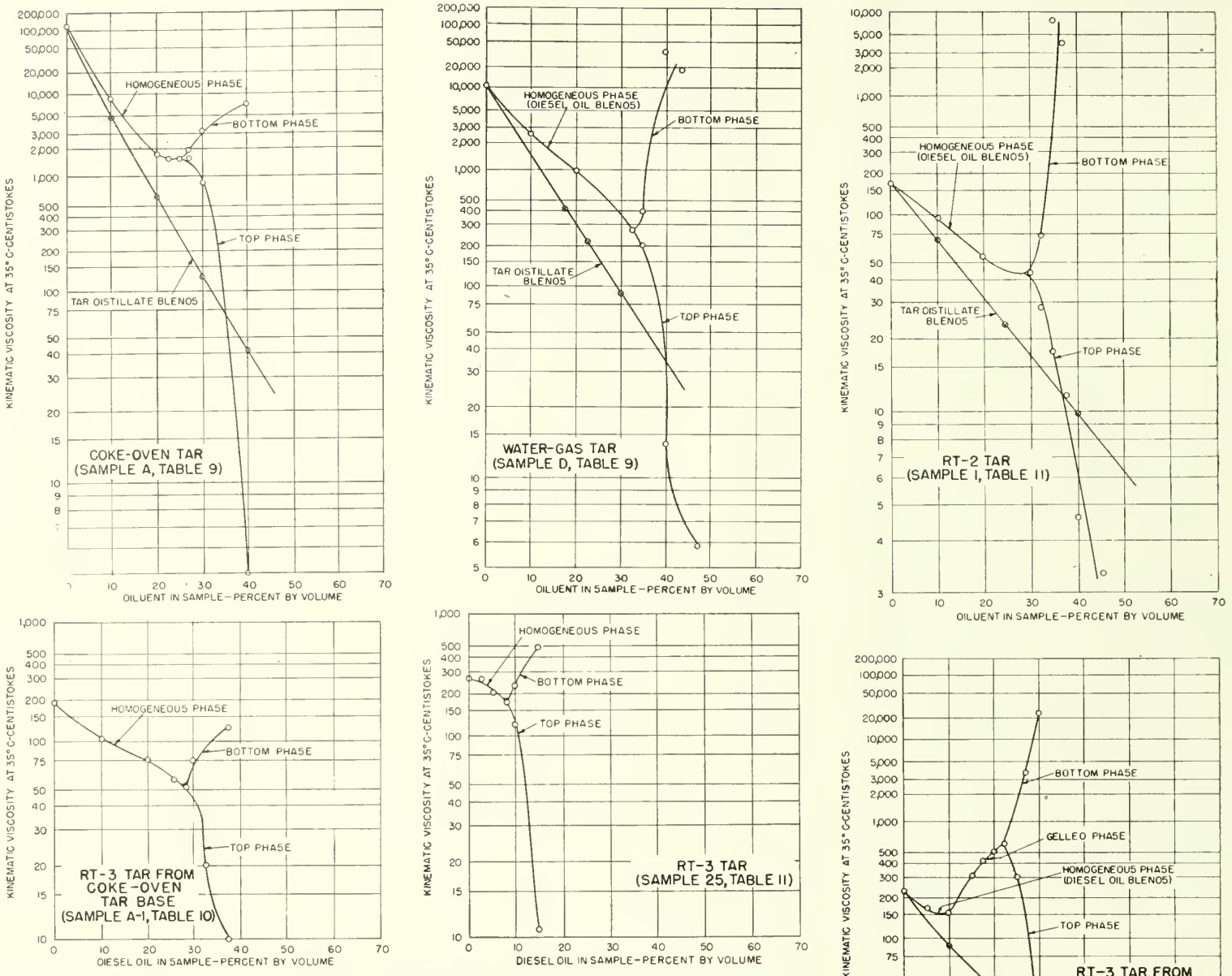

Figure 6.—Effect of various percentages of diluent on the kinematic viscosity at 35° C. of various tars.

soluble in the continuous phase become incompatible and precipitate (precipitation point as shown by microscope).

Essentially the same basic conception of the phenomena involved was discussed recently by E. J. Dickinson.[5] His studies involved a division of the tars into five fractions by a combination of distillation and solvent extraction, and molecular-weight determination on the various fractions. Studies were also made of the effect of these various fractions on the properties of dispersion in several media. His tests led to the following conclusions:

> On this evidence, and from its general viscosity characteristics, it is suggested that tar is an "associated colloid" in which the units are composed of a core of hydrocarbons of high molecular weight surrounded by a solvation layer of hydrocarbon molecules of lower molecular weight; these micelle units are dispersed in a continuous oily medium and there is no abrupt discontinuity of phase between this medium and the nuclei of the micelles.

Dickinson also showed that certain flocculating agents tended to dissolve the protecting media around the micelles resulting in the flocculation of the nuclei. This conception, although expressed differently, is fundamentally the same as that discussed by Oliensis. The progressive flocculation of the dispersed phase may also be explained on the same basis. In the normal tar there is an equilibrium established between the continuous phase and the dispersed phase which is believed to be dependent on the interfacial tension between the two phases. When Diesel oil is added, the surface tension of the continuous phase is changed, which also changes the interfacial tension. This changes the equilibrium conditions, resulting in a coalescence of some of the dispersed particles into larger groups.

[5] *The constitution of road tar*, by E. J. Dickinson; Journal of the Society of Chemical Industry; vol. 64; May 1945; pp. 121–130.

Table 12.—*Comparison of separation point as determined by usual test method and from viscosity curves*

Type or grade of tar	Sample identification	Separation point	
		Usual test method	From viscosity curve
Coke-oven	*A* [1]	27.5	25.0
Horizontal-retort	*B*	22.5	20.0
Vertical-retort	*C* [1]	20.0	25.0
Heavy water-gas	*D*	32.5	32.5
Heavy water-gas (extreme case)	*E*	25.0	22.5
RT-3	*A-1*	27.5	27.5
RT-3	*B-1*	12.5	[2] 10.0
RT-3	*C-1*	32.5	32.5
RT-2	*D-1*	40.0	40.0
RT-3	*E-1*	35.0	32.5
RT-2	*1*	32.5	30.0
RT-2	*8*	27.5	25.0
RT-2	*10*	12.5	10.0
RT-3	*25*	7.5	7.5

[1] Residue from distillation to 270° C.
[2] Based on the point at which the viscosity begins to increase for additional Diesel oil.

U. S. GOVERNMENT PRINTING OFFICE: 1948

Concrete Pavements on the German Autobahnen

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The inspection upon which this paper is based was prompted by a desire to reconcile conflicting reports which have come out of Germany during the last 3 years regarding the performance of concrete pavements on the autobahnen as compared with the performance of similar pavements in the United States. The survey was made during the summer of 1947 and covered approximately 1,000 miles of four-lane divided pavement in the British and American zones of occupation. In this report, the present condition of the German pavements is discussed from the standpoint of both structural performance and quality of concrete per se. All of the structural defects that usually develop in concrete pavements in the United States were found but, aside from frequently observed transverse cracking, defects such as joint spalling and faulting, settlement, etc., were not serious except in the Frankfurt area. The comparative freedom of the German roads from structural defects is believed to be due primarily to the comparatively small amount of heavy truck traffic using these roads, now and in the past, and to the comparatively mild climate.

The soils of Germany vary from cohesionless sands to plastic silty clays and clays. Most of the silty clays were of such nature as to require careful moisture control for adequate compaction. Such soils would be subject to frost heave under adverse drainage conditions, and pumping at joints would occur on them if free water entered expansion joints or cracks and if a sufficient number of heavy loads passed over the pavement. The practice of placing a layer of granular material under the pavement was, no doubt, a contributing factor in the prevention of pumping in such cases.

The concrete, almost without exception, was of excellent quality. Scaling was confined almost entirely to the München–Salzburg route. Disintegration was practically nonexistent. Outstanding was the absence of the heavy layer of surface mortar frequently found on pavements in the United States. The excellent quality of the concrete is believed due to (1) the high quality of aggregates, (2) the low water–cement ratio, (3) thorough consolidation by tamping and vibration of a very dry mixture with a maximum aggregate size of about 1 inch, (4) thorough curing, and (5) the comparatively mild climate. The effect of the cement is not clear—German cements were definitely inferior as judged by modern American standards, but whether they were actually inferior remains to be seen.

As a result of their observations the authors recommend the initiation of a comprehensive program of research on each of the following subjects:

1. Study of the possibility of insuring greater uniformity in pavement concrete by reducing the maximum size of coarse aggregate.

2. Development of more effective methods of compacting pavement concrete by mechanical means.

3. Study of the effects of variations in chemical composition of cements and the methods of manufacturing cements on the properties of concrete. In this work the authors would go considerably outside the range in composition and fineness now being studied under the "Long-time study of cement performance in concrete." Work of this nature should be carried out by the manufacturers and could be accomplished by an extension of the present program of the long-time study.

THIS report presents the results of an inspection of concrete pavements on the German motor-road system made by the authors during the summer of 1947 under instructions from the Commissioner of Public Roads. The inspection was made with the object of determining first, in what respects, if any, the German pavements are superior to ours; and second, what features of German design and construction practice, if any, can be adopted with profit by American engineers.

The report is in four parts: Part I is a general description of the system, including a section on the soils of Germany, a section on climate, and a section on traffic; part II, a description of the methods of design and construction which were employed; part III, comments on the present condition of the pavements with respect to (1) structural de-

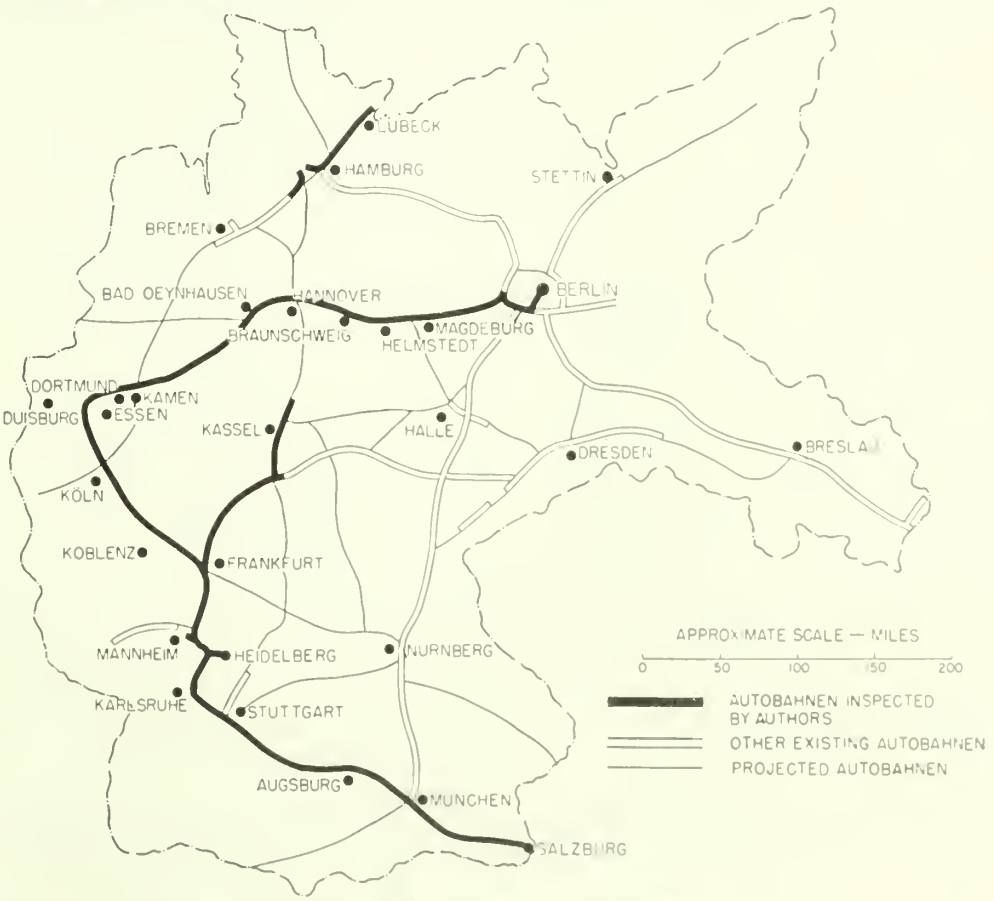

Figure 1.—The German autobahn system.

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fects and (2) quality of concrete per se; and part IV, a summary of these comments, together with a number of recommendations based thereon.

During the 3 years that have elapsed since the invasion of Germany, the system of high-speed motor roads created by Hitler has been the subject of considerable comment by returning members of our armed forces as well as by civilian engineers. Much of this comment has been altogether favorable, not only with respect to the lay-out and design of the system as a whole but also as regards the excellent condition of the concrete pavements themselves. Typical of this reaction is a statement by M. A. Swayze, Director of Research of the Lone Star Cement Corporation, who spent about 3 months in Germany in 1945. In a report presented at the 42nd annual meeting of the American Concrete Institute,[1] he said in part: "In all my experience I have never seen concrete highways in better shape after at least 7 years' service than in the German autobahnen * * * Their quality is a tremendous challenge to American engineers to do likewise." On the other hand there has been some comment to the effect that these pavements, although generally in good condition, are not exceptional when compared with similar pavements in this country in that virtually all of the defects that characterize our roads are present in more or less degree in the German pavements. A statement to this effect was recently made, in an interview with the authors, by an outstanding German engineer who played a rather important role in the construction of these roads, and since that time has had an opportunity to observe conditions in the United States and other countries.

At the outset, the authors wish to thank Dr. W. H. Glanville, Director of the Road Research Laboratory of Great Britain, and his associate, Mr. F. N. Sparkes, for their kindness in making all of the necessary arrangements for this inspection, including the furnishing of automobile transportation for the entire trip. In addition, they wish to express to Dr. Glanville their appreciation for his kindness in detailing two of his staff to accompany them. One of these, Dr. A. R. Collins of the Concrete Section, traveled with the party during the period July 14–August 2, and the other, Mr. H. W. W. Pollitt of the Soils Section, during the period August 2–11. Both of these men were extremely cooperative and their assistance is greatly appreciated.

The authors also wish to acknowledge the assistance of Mr. F. R. McMillan, of the Portland Cement Association, who arranged for the translation of the official instructions for the construction of concrete pavements, and Mr. A. P. Anderson, formerly with the Public Roads Administration, who translated a number of technical reports covering various phases of the work.

[1] *Observations of war damage to concrete and to cement industry properties in Germany*, by M. A. Swayze; Journal of the American Concrete Institute, vol. 18, No. 6, February 1947.

Of the 4,000 miles of four-lane divided highway planned by Hitler, about 2,500 miles had been completed at the outbreak of World War II, as shown on the outline map of Germany in figure 1. On this map the 1,000 miles of roads traveled at least once by the inspecting team are shown as heavy solid lines. Many of the completed roads are in the Russian Zone and were inaccessible—in fact, with the exception of a very short stretch in the French Zone between Köln and Frankfurt, all of the roads inspected by the team lie within the British and American Zones. In addition, the party traveled over the 75 miles in the Russian Zone from Helmstedt to the Berlin ring road, but the cars were not allowed to stop along this route.

The inspection party arrived at Bad Oeynhausen, headquarters for the British Zone, on July 15 and left Frankfurt at the conclusion of the inspection on August 11. Of the 28 days spent in Germany, 10 were taken up in obtaining the necessary clearances, in making other arrangements, and in interviews, leaving 18 days for the actual road inspections. During this 18-day period, the entire 1,000 miles of motor road shown by the heavy solid lines in figure 1 were covered at least once; many sections twice; and several sections, three or more times. Driving speeds averaged 30 to 35 miles per hour, with frequent stops to check soil conditions, quality of concrete, structural failures, and the like. Stops were for periods ranging from a few minutes to an hour and were, in general, made whenever indications pointed to a marked change in soil conditions or in the condition of the pavement itself. The character of the soil profile was checked at many points by means of test holes and outcroppings. Numerous photographs were taken, of which a few are reproduced in this report. Efforts were also made to obtain small samples of the concrete. However, the facilities for obtaining samples were not good and about the best that could be done was to obtain a number of small chips which were mainly of value in revealing the types of aggregate used in the concrete and in giving some idea of the quality of the matrix in each case.[2]

PART I.—GENERAL DESCRIPTION OF SYSTEM

The German motor-road system, as originally planned, apparently contemplated the construction of approximately 4,000 miles of four-lane divided highway connecting all of the larger cities of the Reich. Of this total, only about 2,500 miles have been completed. A glance at figure 1 will show many gaps in the system.

In general, the motor roads were located to pass close to but not actually through the large cities, access being provided in all cases. No crossings at grade were permitted, a cloverleaf or similar type of interchange being used at all major intersections. All secondary roads were carried either over or under the main highway without provision for access. There are numerous bridges for this purpose on the system.

It is not proposed in this report to describe in detail the methods used by the Germans in locating the motor roads or the standards of grade and alinement that were followed. These features have been discussed quite adequately in reports of previous inspections made by British teams.[3] It will be of interest, however, to call attention very briefly to some of the principal standards.

The German motor-road system was designed for speeds up to 100 miles per hour in flat country and up to 80 miles per hour in mountainous regions. Grades varied from a maximum of 4 percent in rolling country to 6 percent in hilly sections and 8 percent in the mountains. Minimum sight distances ranged from 500 to 900 feet, and the minimum radii of horizontal curves from 2,000 to 6,000 feet, depending upon the terrain.[4] The Germans apparently gave a great deal of thought to esthetic consideration, both as regards the original location and the landscaping of these roads. In general, the motor roads were located so as to take full advantage of scenic possibilities. Cut and fill slopes were rounded and were blended in with the natural terrain so as to give a very pleasing appearance. This, together with adequate roadside planting, gave an appearance of age and permanence which is quite surprising when one realizes that the average age is only about 10 years. Typical views are shown in figure 2.

Construction of the motor roads was begun in 1934 and extended through 1940. By far the greatest mileage, however, at least so far as the roads in western Germany are concerned, was built during the 3-year period 1936–38. This applies also to the 1,000 or so miles of pavement inspected by the authors. According to data furnished by the British Road Research Laboratory,[5] the approximate mileages by year of construction for this portion of the system are as follows:

Year built:	*Miles*
1935	70
1936	270
1937	240
1938	325
1939	0
1940	100
Total	1,005

[2] Arrangements have been made with the British Road Research Laboratory to have concrete core samples taken from a number of selected sections on the motor-road system. Samples of the subgrade below each core will also be obtained.

[3] *Report of an inspection of the German road system, 1945*, B. I. O. S. final report No. 918, by H. S. Keep, A. H. D. Markwick, and A. R. Collins, for the British Intelligence Objectives Subcommittee. H. M. Stationery Office, London, May 1946.

[4] The indicated sight distances, nonpassing minimums on divided highways, are low as compared with American standards. The A. A. S. H. O. standard uses sight distances of 475 and 600 feet for design speeds of 60 and 70 miles per hour, respectively. While not included in the standards, corresponding values for design speeds of 80 and 90 miles per hour would be 775 and 1,000 feet, respectively.

[5] According to other records, a short section of road between Frankfurt and Darmstadt, included here as 1935 construction, was actually built in 1934.

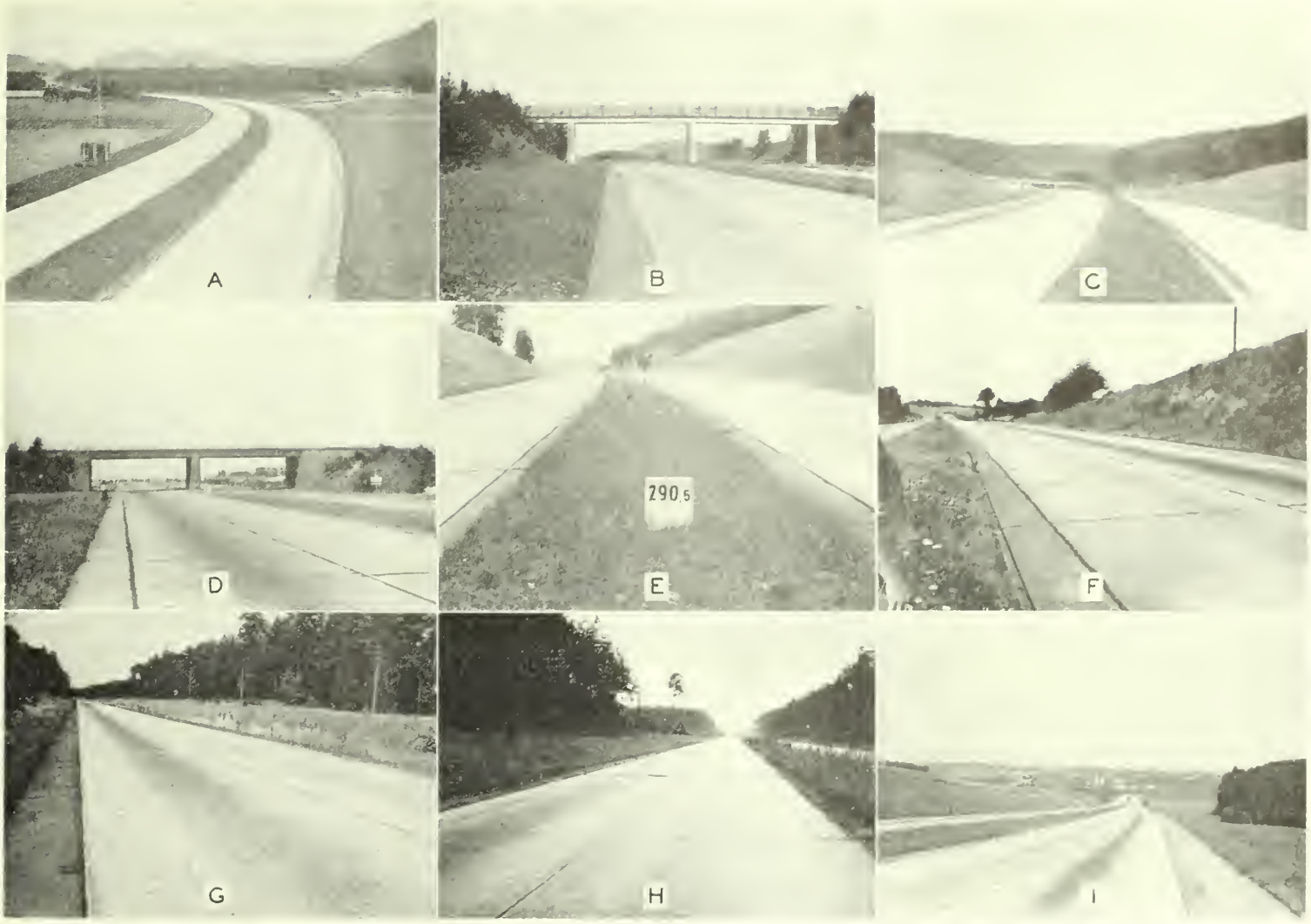

Figure 2.—Typical views of the autobahnen: (A–C) *München-Salzburg;* (D–F) *Hannover-Ruhr;* (G, H) *Frankfurt-Stuttgart;* (I) *Stuttgart-München.*

Soils

The inspection in a limited time of an extended mileage of highways for the purpose of comparing pavement performance with subgrade characteristics requires a regional understanding of soil conditions, especially if the terrain consists of a complex arrangement of geological parent materials with a wide range in relief or climate such as exists in western Germany. In the United States, engineering soil maps prepared from air photographs, geologic maps, or agronomic maps have been useful in developing such a regional understanding. Therefore, prior to leaving the United States, reconnaissance engineering soil maps similar to those shown in figure 3 [6] were made for the terrain in the vicinity of the autobahnen. These strip maps were prepared from geologic, physiographic, and soil information found in available technical publications. They show the major physiographic regions and the general characteristics of the soils in the areas traversed by the autobahnen. Factual data on soils were limited so that the soil information was largely interpreted from data showing the distribution of geologic parent materials, the relief, and the climate of Germany.

The predicted soil information on the strip maps was used during the inspection for the general location of significant soil changes and was useful in evaluating the regional engineering soil problems that probably were considered in locating, designing, and building the autobahnen. Observations made at frequent intervals during the inspection indicated that the information shown in the strip maps was satisfactory for the appraisal of soil conditions in the area traversed by the autobahnen. During the field inspection, the boundaries of the significant soil areas shown on the strip maps were found to be quite accurate. They were checked as frequently as possible by examining soil profiles exposed in cuts, in excavations near the road, or by digging shallow pits.

The inspection revealed that with the exception of a section in which warping was observed there was very little relation between the type of soil and the pavement condition. Defects such as faulting, cracking, and spalling were found about equally in areas where either cohesive or cohesionless soils predominated. This condition is probably the result of placing a layer of granular material over all cohesive soils. The primary purpose of the use of such a layer was to avoid damage to the pavement resulting from frost action on the subgrade soil.

The soils of Germany vary from cohesionless sands to plastic silty clays and clays. Northern Germany is in a glaciated area and the soils are light silty clays, sands, and sand-gravels. Around Berlin and Hamburg, and as far west as Bad Oeynhausen, granular materials are plentiful and were generously used in the autobahnen. From Bad Oeynhausen west to Duisburg, sands and slightly heavier silty clays are about equally distributed. Along the Rhine Valley from Duisburg to Frankfurt are heavier clay soils derived from the metamorphic rocks which predominate in the uplands south of Köln. In several locations in this area granular material could not be found by digging at the edges of the pavement, indicating that scarcity may have limited its use for base courses. This region is somewhat rugged and wooded. From Frankfurt south to Karlsruhe the road follows the Rhine River Valley and crosses the flood plains of several major streams. In this area sands and sand-gravels predominate. The soils in the area from Karlsruhe to Augsburg

[6] These soil maps were prepared by F. R. Olmstead, Senior Soil Specialist of the Public Roads Administration.

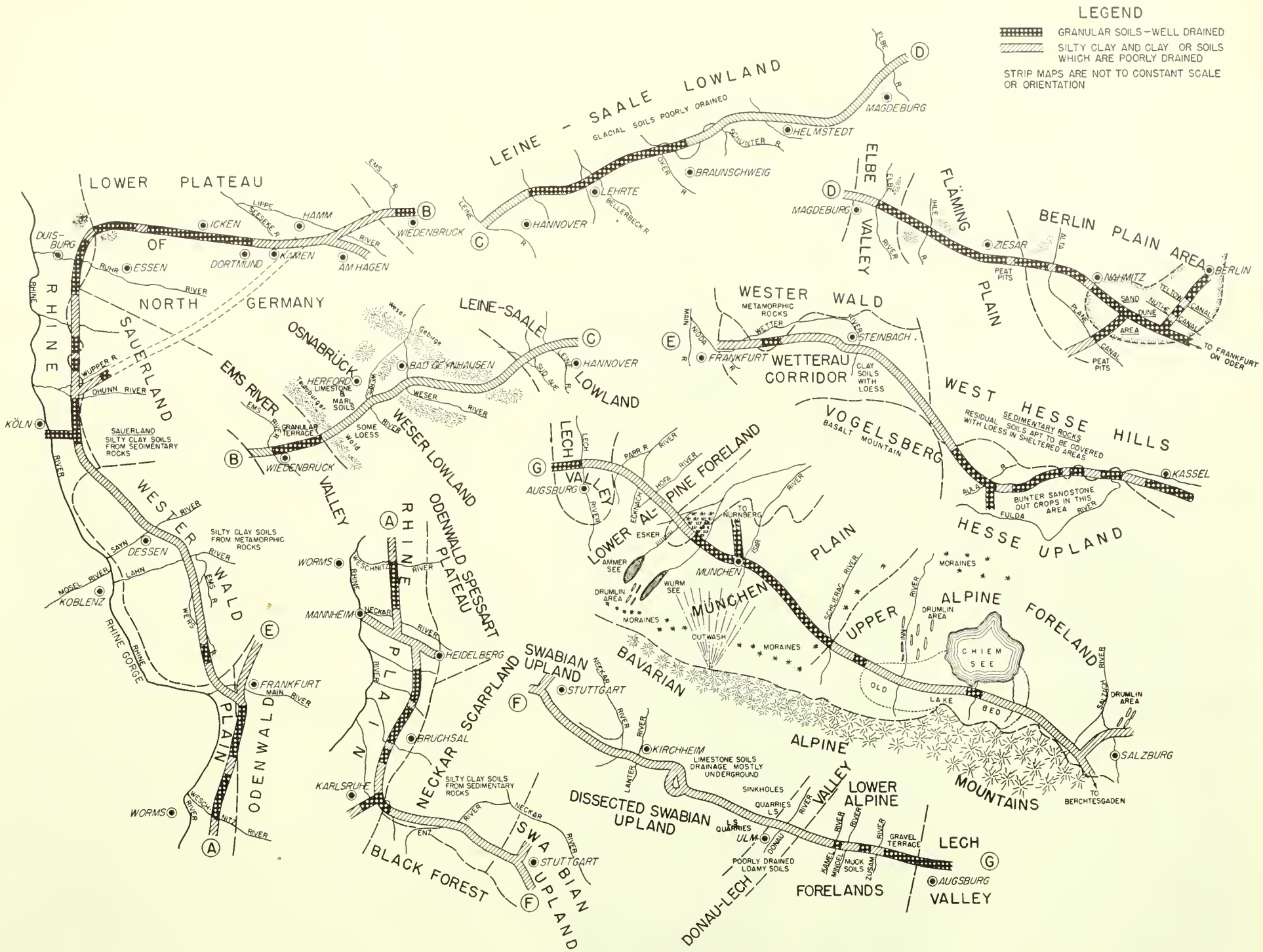

Figure 3.—Engineering soil reconnaissance strip maps prepared from available data prior to the trip.

are derived from sedimentary rocks and, except for the mileage through the gravel terraces of river beds, are plastic silty clays, often mixed with rock fragments.

From München to Salzburg the road passes through a glaciated region and a considerable mileage passes over an old lake bed bordering on the Chiem See (lake). The soils are poorly drained plastic silty clays. Between the Chiem See and Salzburg the road is located along the foothills of the Bavarian Alps and passes over terminal morainic areas. The drainage is poorly established and, in general, the subgrade conditions are poor.

Most of the silty clays examined on the system would require careful moisture control for adequate compaction and would be subject to frost heave under adverse drainage conditions. Pumping at joints in concrete pavements would occur on these soils if free water entered expansion joints or cracks and if a sufficient number of heavy loads passed over the pavement.

The Berlin-Magdeburg section of the autobahnen is located through the area once covered by a continental ice sheet. The glacial till and outwash areas have been modified by subsequent erosion to produce a variety of glacial land forms having different soil textures and drainage conditions. Most of the soils in the region are granular. The area has been separated into three physiographic regions—the Berlin Plain, the Fläming Plain, and the Elbe River Valley.

The Berlin Plain is a flat to undulating lowland with many canals and swamps having intervening sandy areas derived from glacial outwashes, terraces, or sand dunes. The Fläming Plain is gently rolling, with broad low ridges separated by wide flat valleys. Many of the valleys are poorly drained wet meadows, in which the accumulation of peat bogs is frequent. In the Elbe Valley the soils are better drained than in the Fläming Plain, and along the east edge of the river there are many sand dunes.

All of the soil profiles observed in road cuts and excavations in the Berlin-Magdeburg section seemed to indicate that the soils were generally sands. In some locations the sands appeared to be covered with a silty clay surface soil. Since this area is in the Russian Zone, where stopping was not permitted, inspections were made from an automobile traveling at approximately 30 miles per hour.

The section from Magdeburg to Hannover lies in the Leine-Saale Lowlands. The terrain is flat to rolling, with altitudes ranging from 100 to 200 feet. In general, the soils are closely related to the glacial land forms. The granular soils in the region are associated with outwash plains and terraces and the clay soils with the morainic areas. During the glacial period, the Leine and Elbe Rivers deposited vast amounts of alluvium in rather wide flood plains. These were subsequently dissected by erosion to form the present terrain. The better-drained granular soils occur on terraces, and the poorly drained soils on the more recent flood plains. The poorly drained soils appear to be sandy, but drainage is retarded by the development of an iron-oxide hardpan. Peat bogs and swamps are quite frequent in areas underlain by this hardpan formation. In some areas the loamy surface soils are probably derived from wind-blown silts of glacial origin.

An examination of an embankment under repair between Helmstedt and Braunschweig disclosed that the fill was composed of a sand-clay-gravel mixture of good quality, with a layer of very fine white sand immediately under the pavement.

Observations made between Braunschweig and Hannover indicated that the soils were predominately sands and sandy gravels. East of Braunschweig the surface soil was silt to silty loam with indications of underlying sand and gravel.

Geologic and soil information of the area around Hamburg were not available to the authors and a preliminary soil map could not be prepared. The autobahn from Hamburg to Hannover has not been built, and time would not permit inspection of the entire road from Hamburg to Bremen. However, an examination was made of the section from Hamburg to the Hamburg-Hannover-Bremen junction and of approximately 10 miles on the Bremen road. Examination was also made of the Hamburg-Lübeck section.

South of Hamburg the subgrade under a section of pavement which had been broken by a bomb was examined. The soil under the pavement was a fine sand containing a very small amount of binder. The section was on a fill approximately 30 feet in height, and the entire embankment was composed of granular material.

At another location in this area, an examination of the soil profile in a shallow excavation showed 6 inches of fine silt on the surface underlain by fine sand. There was evidence that the sand was coarser at greater depths.

In the fields at one location peat was being dug for fuel. On the south part of the section from Hamburg to Lübeck, the soil, exposed in shallow excavations adjacent to the road, was very sandy. On the north end of this section the soils were silty clays underlain by sands.

Discussion of the construction of the autobahnen with German engineers indicated that the peat-bog areas were carefully avoided in locating the entire system. A few short sections are located over such areas, and the embankments over these locations were built by superimposing the fill material and displacing the muck soil with dynamite charges.

The section from Hannover to Wiedenbrück passes through the Osnabrück-Weser Lowlands and a portion of the Ems River Valley. This area is underlain by beds of limestones, greensands, marls, and clays. The more resistant limestones and greensands occur in low-lying ridges, and the softer marls and clays are found in the valleys of the Weser and Werre Rivers. The Ems River Valley is underlain by these softer rocks but during the glacial period, when the Ems served as one of the glacial drainage outlets, the valley floor was covered with a thick layer of alluvium. The soils occurring in this area, with the exception of those found in the valley of the Ems, are mostly clays derived from the weathering of the underlying calcareous rocks. The granular soils found between the Ems River and the east valley wall occur on an old glacial terrace. In more protected areas, there may be soils derived from remnants of the old glacial drift or deposits of wind-blown silt.

The soil profiles exposed in the fields at most of the points examined between Bad Oeynhausen and Hannover, and to approximately 25 miles west of Bad Oeynhausen, consisted of a gray, silty-clay surface soil (6 to 12 inches) over a more plastic, yellowish silty-clay subsoil.

The practice of hauling granular materials relatively long distances for use in fill construction is illustrated by an observation made at a location about 1 mile east of the Bad Oeynhausen interchange. The embankment at this point was approximately 12 feet high and was composed of cohesionless sand which, it was estimated by visual inspection, would pass a No. 10 sieve and be retained on a No. 100. The sand was covered with 10 inches of friable silt surface soil and a very heavy tough sod. The slope of the fill was approximately 1½:1. A local resident informed the party that the sand for the fill was brought from a pit about 4 miles from the road.

Approximately 30 miles east of Bad Oeynhausen, where the soil profile was exposed in a limestone quarry, the surface soil consisted of a silty clay and the underlying layer was a plastic clay.

At a location 1.5 miles west of the Bad Oeynhausen intersection, an examination was made where the road passes through a sidehill cut. In a pit dug in the median strip, top soil was found to a depth of 10 inches—slightly below the bottom of the pavement. Below this depth was a mixture of sand and stone particles for a depth that could not be determined due to hardness of the ground. An 18-inch pit dug in the embankment contained silty-clay soil. There was no evidence that the granular material extended the full width of the grade. At another location in cut section, 3 inches of cinders were found below the bottom of the pavement over a silty-clay soil mixed with rock particles.

Evidence of substantial sand layers under the pavement and for the full width of the roadway was found at most of the locations examined in this section.

Observations made in the southwest 15 miles of the Hannover-Wiedenbrück section disclose the predominance of sandy top soil underlain by cohesionless sand.

In the section from Wiedenbrück to Dortmund the soils are clays and silty clays derived from the underlying calcareous sedimentary rocks of the low plateau of northern Germany. Except in places where the sedimentary rocks have been exposed by the erosion of the overlying glacial drift, the soils in the section from Dortmund to Duisburg are granular.

Between Duisburg and Köln, granular and fine-grained soils occur in alternate sections of approximately 10 to 20 miles length. The granular materials are located in the Rhine Plain and the silty-clay to clay soils where the road crosses the Sauerland. The Sauer-

land is a rolling sandstone and shale plateau sloping westward and cut by many narrow, steep valleys. The soils derived from these sedimentary rocks usually are silty clays and clays, and are likely to be shallow in depth.

At a location approximately 3 miles west of Kamen, the soils found in an embankment slope, at the toe of the slope of the embankment, and in the shoulder near the pavement edge to a depth of 18 inches, were gray, friable, silty clays and yellow, silty clays. No evidence of a granular layer under the pavement was found by digging shallow pits in the shoulder at this location. It is possible that the granular material was not placed for the full width of the paved shoulder strip.

At another location, approximately 10 miles south of the Ruhr River, a 6-inch layer of crushed shalelike rock (maximum size ½ inch) was found beneath the pavement slab. The subgrade soil was a yellowish, mottled, plastic clay. The clay had intruded into the stone subbase and the pavement at this location was rough and the joints had the appearance of being warped.

In some other locations examined, evidence of the use of granular material over plastic clays could not be found by digging near the edge of the paved shoulder strip.

The section from Köln to a point west of Frankfurt, on the River Main, passes through the lower Rhine Plain and the Westerwald. The Westerwald is a folded, contorted mass of schist, quartzite, and other Devonian rocks. The soils derived from these rocks are clays and sandy clays. The soils in the 20 miles immediately south of Köln or to the boundary of the Westerwald were found to be granular. The soils in the remaining mileage of this section were silty clays or clays. No extremely plastic soils were found in this section.

From Frankfurt to Karlsruhe the road follows the terraces of the Rhine Rift Valley. The soils in this valley are usually granular but may be covered with a shallow layer of silty-clay soil. From the Enz River (near Karlsruhe) to Stuttgart is an area of calcareous soft rocks or marls. The soils in this area are clays or clays mixed with stone fragments.

Observations made between Frankfurt and Heidelberg indicated 5- to 10-mile sections of cohesionless sand covered with sandy surface soil, interspersed with sections of clay and silty clay similar in length. In the areas where the clay soil predominated, the fills were built of cohesionless sand.

From Karlsruhe to Stuttgart, silty-clay and clay soils predominate. In some locations the clay soils seemed to be quite plastic. Granular material could not be uncovered by the digging of shallow pits in the embankment slopes in this area. Pits dug at the edge of the pavement in the median strips disclosed a layer of either fine sand or sand-gravel under the pavement for a minimum depth of 12 inches.

From Stuttgart to München the road traverses the Swabian Upland, the Donau–Lech Valley and the Alpine Foreland. The Swabian Upland is flat to gently rolling and is dissected by streams with steep valley walls. The parent material in this area is limestone and the soils are chiefly clays. The Donau–Lech Valley and the lower Alpine Foreland are glaciated areas. The soils vary from fine-grained, poorly drained alluvium and clay soils to sands and sand-gravels. There are terraces of granular material along the Lech River. Loess soils cover the glacial terrain in sheltered locations.

Between München and Salzburg the road traverses the Münchehen Plain and the Upper Alpine Foreland. The München Plain is a granular outwash developed from water sorting of granular material contained in the terminal moraines that flank the Bavarian Alps to the south of the area. The Upper Alpine Foreland is a glaciated area made up of terminal moraines, drumlins, and an old lake bed. The soils in this section are fine sands, silts, and silty clays mixed with gravel and stone. Between the Chiem See and Salzburg the road passes along the foot of the Bavarian Alps, cutting through limestone outcrops and passing over terminal moraines.

The soils in this area are clays or alluvial silts and the land is swampy in many places. The drainage is poor and there are evidences of distress in the pavement at some locations. The view of the Alps from this section of road is good and it is probable that, in choosing the location, the scenic value outweighed the poor soil and drainage conditions. A location further north would have passed through more suitable terrain.

The autobahn from Kassel to Frankfurt passes through the Hesse Uplands, the West Hesse Hills, the Wetterau Corridor, the Westerwald, and the lower plain of the Main River. The Bunter sandstone and tertiary clays and silts outcrop in the area around Kassel. The road is flanked on the west by the West Hesse Hills and on the east by the Hesse Uplands, which are complex geologically since they were formed by the erosion of faulted and folded sandstones and limestones and the subsequent deposition of loess. The road enters the Wetterau Corridor at a point west of a basaltic mass called the Vogelsberg Mountains which rise to 2,500 feet and which have been dissected by radial stream gorges to depths of more than 600 feet. Many of the stream valleys are filled with peat. The ruggedness of the terrain necessitated the construction of many high embankments which observations indicated were built of cohesionless sand.

The soils in the Wetterau Corridor are heavy clays derived from the weathering of basalt. The soils in the Westerwald and the Main River Plain are tertiary clays and wind-blown loess. The clays were derived from the weathering of the shales and schists of the Westerwald. The loess has weathered to a more plastic silty clay in some areas.

Climate

Western Germany has a milder climate than would be expected from its latitude because it is exposed to mild westerly winds. There is little difference between the north and south sections since in the latter the higher elevations counterbalance the southern location. The mean annual temperature of Hamburg is 48° F., of Leipzig, 47° F., and of München, 45° F. The average annual rainfall varies from 16 inches in the plains to 47 inches on the southeast uplands. July is normally the rainiest month and January or February, the driest. The snowfall occurs during the months of January, February, and March, and in that period snowfall occurs about 30 days in the northern lowlands and 30 to 50 days in the southeastern highlands.

It would be difficult to select an area in the United States having an extensive mileage of concrete pavement in which the soil and climatic conditions are exactly the same as those found in Germany. The area including southern Michigan, Ohio, Indiana, Kentucky, western Pennsylvania, and West Virginia has similar geologic formations, soil conditions, and a fairly comparable climate. The temperature and rainfall data for six cities in the selected area of the United States and for four cities in Germany, as supplied by the United States Weather Bureau, are shown in table 1. A comparison of these data shows that the average temperatures for January are approximately the same in both areas, that the average July temperatures in the United States cities are approximately 10 degrees higher, and that the range from maximum to minimum is greater in the United States. The rainfall in Indianapolis, Columbus, or Pittsburgh is comparable to that of München, but for the rest of Germany it is lower than that of the middlewestern area of the United States.

Table 1.—*Comparison of climatological data for Germany and midwestern United States*

City	Temperature					Annual average rainfall
	Average		Variation [1]			
	January	July	Maximum	Minimum	Range	
Germany:	°F.	°F.	°F.	°F.	°F.	*Inches*
Berlin	30	64	99	−15	114	23
Hamburg	32	63	92	−6	98	29
Frankfurt	32	66	100	−7	107	24
München	28	64	97	−14	111	36
Average	30.5	64.2	97	−10.5	107.5	28
United States:						
Indianapolis, Ind	28	75	107	−22	129	38
Columbus, Ohio	31	75	106	−20	126	34
Pittsburgh, Pa	32	74	103	−20	123	35
Charleston, W. Va	38	77	108	−17	125	46
Frankfort, Ky	36	78	111	−17	128	43
Lansing, Mich	23	71	102	−26	128	31
Average	31.3	75	106.2	−20.3	126.5	37.8

[1] Over a period of 35 to 75 years.

Data as to the penetration of frost in Germany were not available from the weather records. It was learned from a German engineer, now working in England, that the frost penetration varied from a few to 70 or 80 inches.

Traffic

The traffic on the autobahnen observed during the inspection was very light both as to the number of vehicles and the probable axle loads. For example, in traveling during the middle of a Sunday forenoon from Helmstedt to the outer Berlin circle, a distance of 70 miles, no vehicles were seen in a period of 45 minutes. Six light trucks were counted in the 70 miles. In many places in this area and other sections of the autobahnen, the pavement had not been discolored by the oil streak parallel to the center line which usually results from the passage of a large number of heavy motor vehicles. The appearance of the pavement indicated that the greatest volume of traffic on the system was from Frankfurt to Kassel, south of Frankfurt toward Heidelberg, and from Frankfurt to Hannover via the Ruhr Valley. Due to the very light traffic on the system, now and in the past, it is difficult to make any comparison of the pavements with those in the United States laid under similar climatic conditions and on similar soils. It is likely that the volume of traffic on most of the primary roads in the United States is considerably greater than was observed at any location on the autobahnen.

PART II.—DESIGN AND CONSTRUCTION

Earthwork and Subgrade

In the location and planning of the autobahnen, detailed surveys were made to evaluate soil and drainage conditions. Whenever possible, bad subsoils were bypassed. Where such areas could not be avoided, detailed investigation was carried out to determine the magnitude of the settlement of soft undersoils due to loading, the time required for such settlements to occur, and the most economical height and slope of embankments. When it was necessary to place high embankments over yielding subsoils, the completion of the earthwork was planned so as to permit the major portion of the settlement to take place before the concrete slab was placed. The course of the settlement was accelerated through lateral displacement of the subsoil by overloading or by blasting.

In the design of the earthwork, danger from frost damage was given special attention. Soils which contained more than 3 percent of particles smaller than 0.02 millimeter were classified as subject to damage by frost either by heaving or by softening. It was recognized that water must be able to reach the frost zone by capillarity before frost damage could occur, and if the ground-water surface was at a greater distance below the frost zone than the capillary rise in the soil, no frost damage would result provided there was no entrance of water from the shoulders or side slopes. Efforts were made to avoid frost damage by the following means:

1. Placing the concrete slab a sufficient height above ground water to avoid saturation by capillarity, or lowering the ground-water table by drainage.

2. Replacement of soils susceptible to frost damage with soils not susceptible to such damage.

3. Use of layers of coarse granular material or of impervious strata such as bituminous concrete slabs, to prevent capillary rise of ground water.

The following statement, from a publication entitled "Guiding Principles for Roadway Slabs" issued for the use of engineers engaged in the design and construction of the autobahnen, illustrates the importance placed on investigation of possible frost damage: "The prevention of frost damage must in every single case be exhaustively investigated according to terrain and change in soil, and may not be handled in a routine manner."

In areas where ground water was found to flow toward the road, it was intercepted by longitudinal drains. Drains were also used to lower the ground-water elevation under the roadway when such a procedure was considered effective and economical.

In a great majority of the mileage, the concrete slab was placed on embankment. In flat terrain the grade line was raised 3 feet or more above the natural ground level, and in cuts the roadway was excavated below grade and backfilled with selected material. This procedure provided good drainage and in the open plains simplified snow removal.

Cohesionless sand was used in fills wherever it was available. Sand was hauled long distances to avoid the disturbance of farm lands that would result from taking borrow with a shorter haul. When sand was not available for the entire embankment, it was used in the top 8 to 32 inches. A thickness of 8 inches was most common. The sand was placed the full width of the grade in fill sections.

Where sand for the entire fill was not available, relatively thin layers of granular material were placed at several elevations and to the full width of the embankment.

Embankment slopes were made steep (usually 1½:1 or steeper) to reduce to a minimum any encroachment on farm land, and embankment slopes were protected against the entrance of moisture by sodding immediately after completion. In some instances a full-width layer of granular material was placed in the bottom of embankments composed of cohesive soil, to aid in the drainage.

Cohesionless sands were compacted with a rammer called a "jumping frog." The device

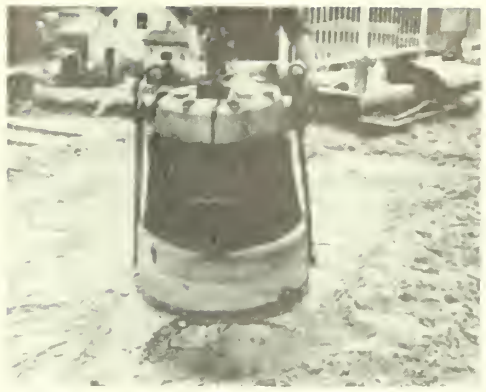

Figure 4.—"Jumping frog" used to compact cohesionless sand.

was actuated by explosions of gasoline in a cylinder and the explosions were controlled by a manually operated spark. A photograph of one of these machines taken by F. R. McMillan is shown in figure 4. The weights of the "frogs" for embankments varied from ½ to 1 ton and for the area under the paved shoulder a machine weighing 800 pounds was used.

Cohesive soils (silty clays and clays) were compacted with smooth-face and sheepsfoot rollers. The thickness of each layer of soil before rolling was approximately 18 inches. A field laboratory was maintained on each project where cohesive soil was used, and the moisture and density of the soil were carefully controlled. Very little effort was made to compact sand or cohesive soil back of bridge abutments and, as a result, there are many settlements in the approaches to bridges. That such settlements were anticipated is evidenced by the fact that stone setts were placed adjacent to many bridges so that fill settlement could be corrected conveniently. The setts were arranged in a fanlike pattern similar to that employed in the case of Durax block in this country.

Shoulders

The use of paved shoulder strips, described in detail in the next section of the report, has been effective in keeping the traffic on the pavement. As a result, there has been no rutting or destruction of the sodded area of the shoulders. The sod in all cases observed was very thick and tough and even though there has been very little mowing or other care given to the shoulders or embankment slopes, they were in good condition and looked reasonably neat. Typical views of the shoulder strips are shown in figure 2.

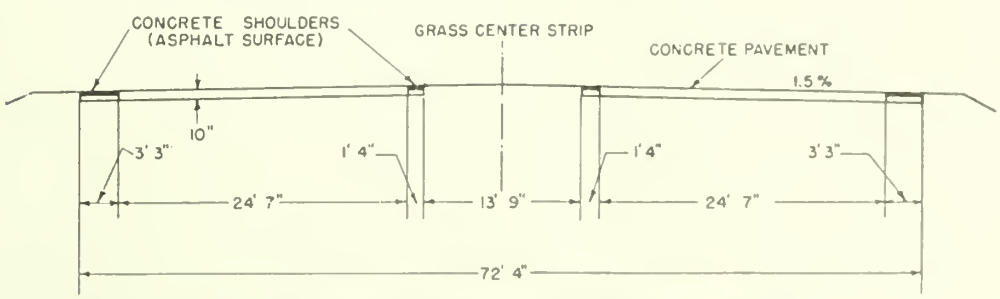

Figure 5.—Typical cross section of four-lane autobahn.

Figure 6.—Where dark-colored pavement was used, the shoulder strips were of plain concrete for contrast.

Pavement Design

With few exceptions, the entire system was designed to provide for a four-lane divided highway, with opposing traffic separated by a grass center strip about 13 feet wide. A cross section of a typical design is shown in figure 5. In this figure and throughout the text, the metric dimensions used by the German engineers have been converted to their approximate English equivalents. Pavements were uniformly 24 feet 7 inches wide, with a longitudinal center joint. Construction, in general, was full width in one operation, with a longitudinal joint of the weakened plane type. Lane-at-a-time construction was occasionally used, the longitudinal joint between the slabs in this case probably being of the butt type, although information on this detail is lacking. Light steel tie bars were used across the longitudinal joint to prevent separation of the pavement slabs due to settlement or other causes.

All pavement slabs were of uniform depth, 8 or 10 inches in thickness, although heavier sections were used occasionally. A unique feature of the design was the hard-surfaced shoulder strips. These were usually 1 foot 4 inches wide on the inside shoulder and 3 feet 3 inches wide on the outside. On later work, however, the outside width was increased to 7 feet 4 inches, apparently with the idea of providing more adequately for emergency parking. The shoulder strips were usually about 10 inches in depth, the lower 7 inches usually being of concrete and the upper 3 inches of asphalt. The purpose of the asphaltic surface was to provide a contrast in color and thus define the outside edge of the traffic lane. In cases where dark-colored concrete or asphalt was used in the pavement proper, plain concrete was used in the top course of the paved shoulder strip (fig. 6).

The concrete base for the shoulder strip was constructed before the pavement slab and was utilized as a base upon which to mount the rails carrying the heavy construction equipment used in the mixing, placing, and finishing operations. The asphalt top was placed to the elevation of the finished roadway after the completion of the pavement proper.

So far as the design of the slab itself is concerned, practice appears to have been patterned largely after that followed in this country, except that the Germans did not use the thickened-edge section. Practice as regards transverse expansion-joint spacing varied widely. For nonreinforced sections the joint spacing was generally indicated as 26 feet, whereas for reinforced sections spacings varied from a minimum of 39 feet to a maximum of 98 feet. Observations at the time of the inspection indicated an average expansion-joint spacing of from 40 to 60 feet. It was also quite common practice to systematically increase the joint spacing from slab to slab in increments of 3 feet or so, with periodic return to the first spacing, apparently in an effort to avoid setting up rhythmic vibrations in cars traveling at constant speed. Weakened-plane contraction joints were frequently installed, particularly in the case of the longer expansion-joint spacings.

Load transfer was provided by means of conventional round steel dowels, 14 inches long and ⅞ inch in diameter, with a sleeve over one end. One typical design for an 8-inch section shows the dowels spaced 12 inches apart at the center of the slab with somewhat closer spacing near the edge. In this design the expansion-joint opening is shown as ¾ inch at the top, with a wood filler ½ inch in thickness.

Practice as regards the use of reinforcing steel also varied. It was apparently not used at all in 1935 but was employed quite extensively in 1936, mostly in the form of a light mesh weighing from 42 to 51 pounds per 100 square feet. Steel mesh was usually placed after screeding the lower course of concrete and before placing the upper or wearing course. This placed the steel at about one-third the depth of the slab, or about 3 inches from the top. Use was also made of corner- and edge-bar reinforcing during this period. The use of reinforcing was discontinued in 1937 and thereafter, due to a shortage of steel.

Materials

Mr. M. A. Swayze's trip to Germany in 1945 was made for the specific purpose of studying the German cement industry. With Mr. G. G. J. Davis, representing the portland cement industry of Great Britain, he visited some 28 cement plants, all located within the American and British Zones. The results of their survey have been reported in considerable detail elsewhere,[7] but are summarized by Mr. Swayze in his report on war damage in Germany, previously cited, as follows: "The great majority of the plants visited were making a product which was inferior in practically every way to the cements manufactured in the United States or in Great Britain. Their control of raw mix proportions was generally lax, and fine grinding of these materials which we believe essential for good quality was nowhere near that of American practice. The burning operation is much lighter than ours, even in plants with modern kilns, and free lime contents as high as 2.5 percent are not uncommon."

Mr. Swayze's appraisal was, of course, based on observations of conditions in 1945. To just what extent economic conditions, particularly an acute fuel shortage, may have tended to lower postwar manufacturing standards as compared with those in effect during the thirties, cannot be stated definitely. Furthermore, the cements used in the construction of the motor roads were supposed to meet special requirements for flexural strength, drying shrinkage, and fineness which were not imposed in the case of the ordinary product. Even so, it seems probable that even these specially selected cements were quite inferior, when judged by our standards, to modern American cements. For example, Swayze's survey indicated that, entirely aside from the matter of fuel economy, the methods for controlling uniformity of product, such as facilities for storing and handling clinker, proportioning materials including the gypsum, etc., were very crude as compared to modern American practice. This conclusion would, of course, apply as well to the prewar German cement as to the postwar product.

The autobahn cements were undoubtedly somewhat coarser than our present-day American cements. The Germans required that between 5 and 25 percent be retained on a sieve having 178 openings per inch. Compare this with the modern American cement, which rarely will have more than 3 or 4 percent retained on the standard No. 200 sieve (200 openings per inch). Particle-size distribution in the typical autobahn cement was also probably quite different. According to Swayze, practically all finish grinding in German cement mills is by open circuit as compared with our modern closed-circuit operation using air separators. In open-circuit grinding, the final cement product contains the various sizes just as produced in one continuous passage through the grinding mills. In closed-circuit grinding, the various sizes are classified by means of air separators and the coarser particles are returned to the mills for further grinding.

It is probable also that the German cements were not as hard burned as our modern cements. Apparently the only requirement for soundness was a 2-hour boiling test. The autoclave test has never been used. According to the best technical thought in this country, an autoclave test is absolutely essential for the proper control of soundness. Many failures in the United States, particularly in the southeast, have been attributed to the absence of an autoclave requirement in specifications, and it will be interesting to note when discussing the present condition of the autobahnen that such a requirement has never been used by the Germans.

According to Prof. Otto Graf,[8] cements for concrete road construction were selected from the commercially available products, on the basis of the following characteristics:

[7] *German cement industry*, F. I. A. T. final report No. 519, by M. A. Swayze and G. G. J. Davis, for the Field Information Agency, Technical, United States Group Control Council for Germany; published by the British Intelligence Objectives Subcommittee. H. M. Stationery Office, London, November 20, 1945.

[8] *Choice of cements for concrete road construction and some questions regarding same*, by Otto Graf; Zement, vol. 25, No. 33, Aug. 13, 1936.

1. High flexural strength.
2. Moderately high (adequate) compressive strength.
3. Low drying shrinkage.
4. Moderately coarse grind.

In addition, as mentioned above, all cements were required to pass a 2-hour boiling test for soundness. This was a field control test for uniformity during manufacture, the other tests being used primarily as prequalification tests to determine the acceptability of a particular source.

Apparently no attempt has been made in Germany to classify cements by chemical composition in a manner similar to that employed in the five-type specification recognized by the American Society for Testing Materials. So far as could be determined, any cement that would qualify on the basis of the physical tests outlined above would be acceptable as to chemical composition on much the same basis as the Type I cement in the A. S. T. M. specifications, with one exception —the Germans placed an upper limit of 3 percent on magnesia instead of the 5 percent used in most American specifications. As a matter of information it should also be noted that the alkalies (sodium and potassium oxides) in German cements are said to run quite low as compared to many American cements.

In addition to straight portland cement, the Germans recognized two types of portland-blast-furnace slag cement: Eisenportland, containing about 70 percent portland cement and 30 percent blast-furnace slag, and Hochofen cement, containing about 15 percent portland cement and 85 percent blast-furnace slag. Because of its slow hardening characteristics, the Hochofen cement was not used to any extent in road work. The Eisenportland, however, was used quite extensively, particularly in northern Germany. The manufacture of this type was, of course, limited to those plants sufficiently close to the blast furnaces to make the utilization of the slag economically justifiable. The availability of almost unlimited quantities of slag, together with the great saving in fuel, made the manufacture of Eisenportland very attractive; and it is understood that in the neighborhood of one-third of the entire mileage of motor roads contains this type of cement.

The instructions governing the selection, processing, and control of aggregates for concrete pavement construction were such as to insure excellent results from the standpoint both of the quality of the hardened concrete and the uniformity of the consistency of the mixture as it was placed. Aggregate quality was insured through the use of compressive strength and wear tests in the case of crushed rock and wear tests in the case of gravel. Hard rocks, such as granite, basalt, and quartzite, were preferred. Durability was judged apparently on the basis of experience, as the record does not indicate that laboratory tests for soundness were used. Blast-furnace slag was permitted but the inspections did not reveal that it had been used to any extent. The requirements for blast-furnace slag were stated to be about the same as for crushed rock. Considerable attention was paid to the matter of impurities, and freedom from flat and elongated particles was insured by requiring that the breadth of an individual particle should be not less than 65 percent of its length and that its thickness should be not less than 25 percent of its length.

In the matter of separated sizes, German practice differed markedly from that employed in this country in that it was apparently the universal practice to require that the sand or fine aggregate be furnished in at least two fractions, 0–0.12 inch and 0.12–0.28 inch. To insure better control of grading, the material below 0.12 inch was sometimes still further separated into two or three sizes. It is obvious that with so many separations, very close control of the total fine aggregate grading could be obtained.

For the larger sizes, the separations were usually 0.28–0.59 inch and 0.59–1.18 inches. These were the size separations for the top course. In two-course construction, a somewhat larger (1.58–1.97 inches) maximum size was used for the bottom course.

Most of the pavements on the autobahnen were constructed in two courses. This was necessary not only because of the difficulty of properly compacting a single course of very dry concrete 8 to 10 inches in depth, but also because this type of construction made it possible to use a somewhat inferior type of aggregate in the lower course—a matter of considerable economic importance in some instances.

Composition of Mix and Proportioning

It was the general practice to design all paving mixes in the laboratory prior to construction, using the same cement and aggregates to be furnished later to the job. The basis of the design was the development of a mix that would (1) have sufficient workability for placing and finishing with the equipment to be used on the work, and (2) result in concrete conforming to the following requirements for strength at 28 days, using a cement content of not less than 1.35 barrels per cubic yard and not more than 1.57 barrels per cubic yard:

Flexural Strength:	*Lbs per sq. in.*
Minimum	540
Average	640
Compressive Strength:	
Minimum	4,700
Average	5,700

It was further provided that the strengths at 7 days should not be less than 70 percent of the above values. It was the practice also to check the strength design by means of tests of compression and flexure specimens made during construction and by means of tests on cores made at the age of approximately 60 days.

As previously stated, aggregates were supplied in a number of separated sizes. Instructions for the design of mixes required that the available sizes be so proportioned that the combined grading curve would fall within the limits shown by lines *K* and *D* in figure 7. The instructions also stated that the exact position of the combined grading curve between these limits would depend upon the nature of the aggregate (that is, the curve would be lower for gravel than for crushed stone) and also upon the efficiency of the placing equipment. In general, a curve below line *J* but above line *D* would be permitted only in cases where compacting machines of proved efficiency were to be employed.

The amount of mixing water apparently was not specified directly but was indicated as the minimum required for the chosen method of placing. For the zero-slump concrete which seems to have been used normally, a water-cement ratio between 0.35 and 0.50, by weight, was specified. Apparently a somewhat higher water-cement ratio was permitted when the concrete contained crushed stone than when gravel was used.

Paving Operations

In contrast to design practices, the construction methods used by the Germans were

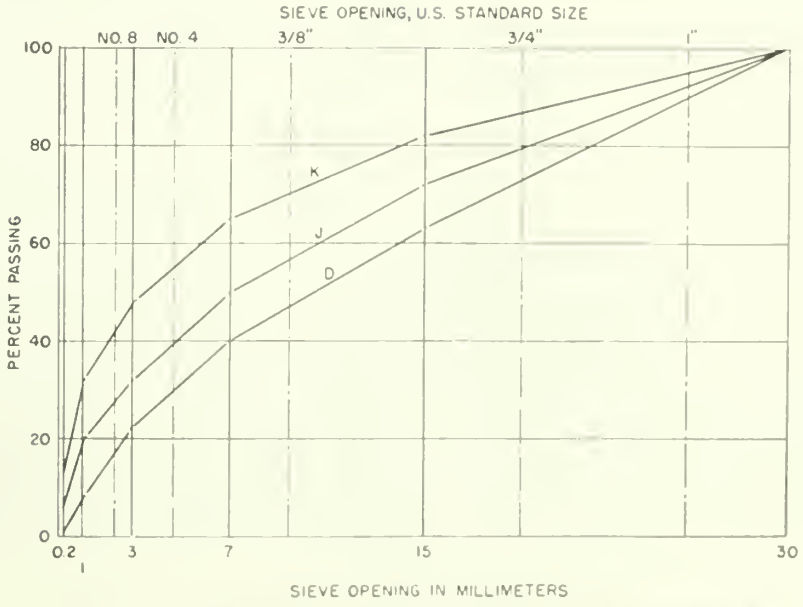

Figure 7.—Grading curves for proportioning combined aggregate.

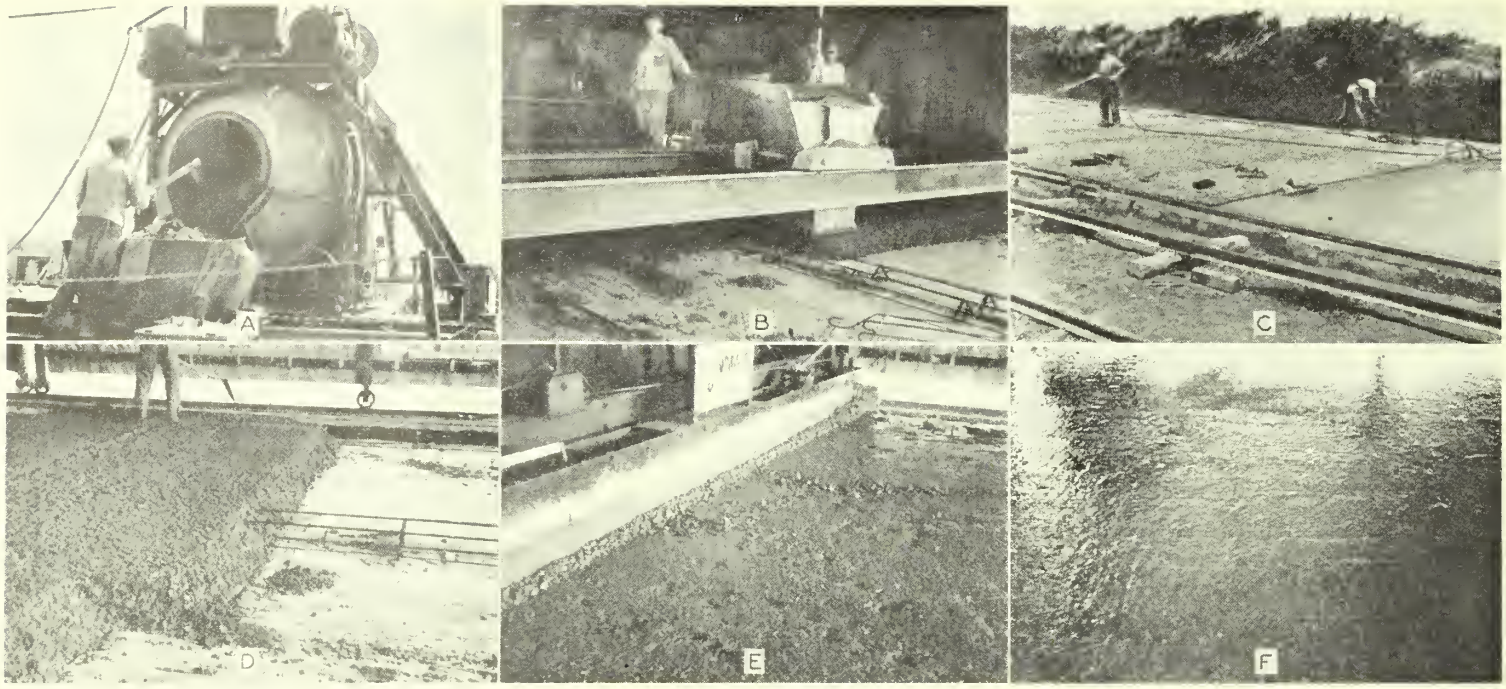

Figure 8.—Construction on the autobahn south of Stuttgart: (A) *Dumping concrete into spreader hopper;* (B) *spreading concrete;* (C) *watering curing mats. Texture of the concrete is shown* (D) *after spreading,* (E) *as the Vögele vibrator began a second pass, and* (F) *after the final finishing.*

totally different from those employed in this country. With minor exceptions it was the German practice to use trains of heavy traveling equipment in connection with the various paving operations on the autobahnen. This equipment, spanning the entire 25-foot-wide pavement slab, was carried on steel rails which, in turn, were mounted on the concrete shoulder strips previously mentioned. Separate mixers, spreaders, and finishers were used for each course. Mixers traveled on the rails and not on the subgrade and were serviced by industrial railways running on the center strip. Proportioning was at a central plant, each size of aggregate being weighed separately. The mixer was sometimes mounted separately and sometimes on the same frame as the spreader. Spreaders were of the vertical-discharge hopper type. After receiving a load of concrete they were discharged by moving the hopper slowly from one side to the other as the entire machine moved forward.

It was the general practice to use a very dry (zero-slump) mix in pavement construction. In this respect the Germans followed British practice rather than that favored generally in this country. Such mixtures are, of course, entirely unsuited for use with an ordinary power-driven screed, which exerts comparatively little compactive effort but merely smooths the surface after the concrete has been distributed by the spreader. German engineers have developed several types of compaction equipment. In fact, considerable research appears to have been conducted on the problem, the results of which are

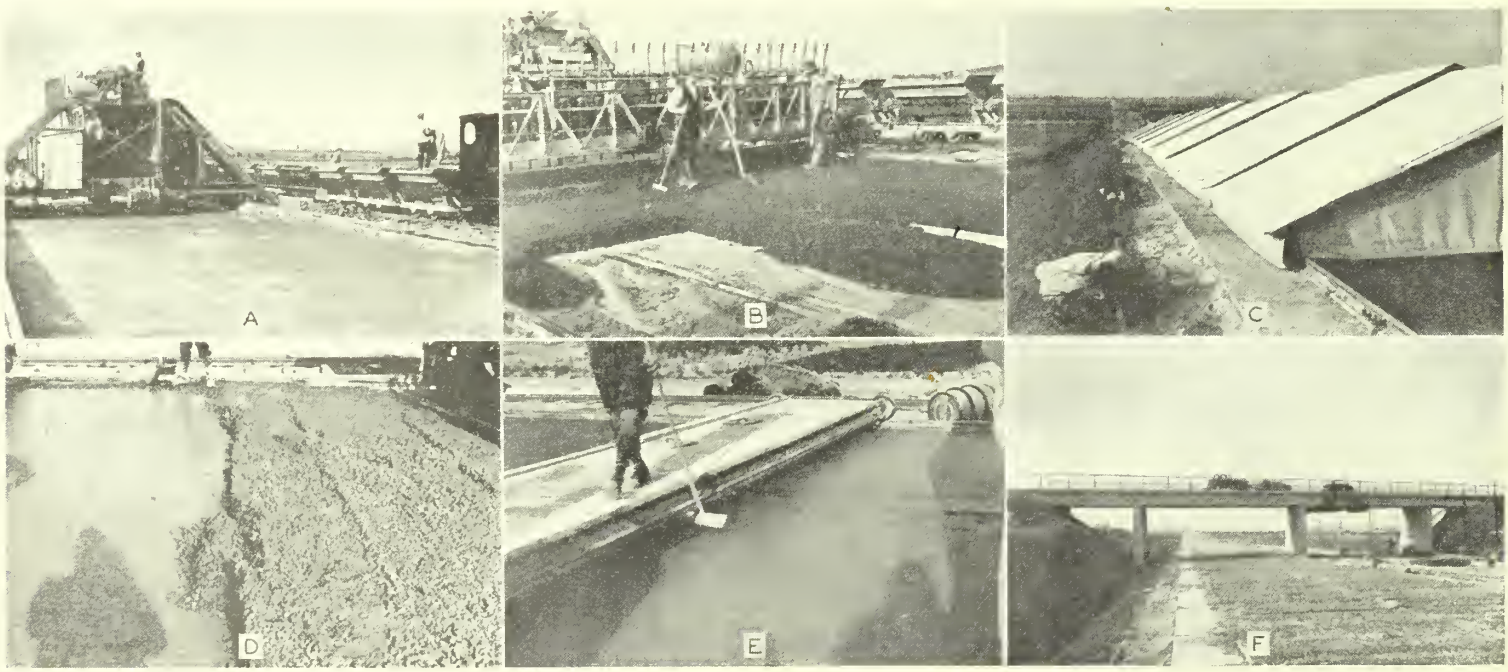

Figure 9.—Construction on the autobahn north of Kassel: (A) *Mixer spanning subgrade, with industrial railway alongside;* (B) *Dingler tamper in operation;* (C) *sunshades used for preliminary curing. Texture of the concrete is shown* (D) *before and after tamping, and* (E) *after brooming.* (F) *Woven-reed curing mats were used.*

discussed in detail in a report by Schade originally issued in 1940.[9]

There were two general types of finishers used on the motor roads, the Vögele and the Dingler. The Vögele was a three-element machine, the first and third elements being reciprocating screeds of the usual type. Mounted between the screeds was a heavy plate which could be operated either as a vibrator or a tamper. Vibrating frequencies were originally quite low (150–250 per minute) but were later increased to as high as 3,600 per minute. It was sometimes operated alternately as a tamper and as a vibrator in successive passes, and frequently several passes were required to obtain the desired finish. The Dingler machine was a very much heavier piece of equipment. It consisted of a series of tamping hammers, with 4- by 10-inch faces, weighing 55 to 120 pounds each and mounted in a row just behind the front screed. The height of fall was about 6 inches and they were operated at about 70 blows per minute. These tamping hammers were apparently only used for compacting the lower course. The final finishing was by means of a combined screed and tamper operating at about 600 strokes per minute and several passes were usually required to produce the required finish.[10]

Another unique feature was the practice of conducting the placing, finishing, and preliminary curing operations under sunshades. These were mounted on light wooden frames spanning the entire pavement and carried on light steel rails mounted outside and parallel to the concrete shoulder strips. In this way the concrete was protected from sun and wind from the time it was placed until it had hardened sufficiently to receive the final curing mats. The frames used for protecting the concrete during placing were sufficiently high to permit the equipment to pass under them and to accomodate the men working on the final finish. The frames used for the preliminary curing were low and were completely enclosed.

During the summer of 1936 Mr. F. R. McMillan, of the Portland Cement Association, visited Germany and took a number of photographs of the various paving operations in use that year. The authors have his very kind permission to include in this report several of these photographs, which illustrate in a striking manner the marked difference between the construction methods used by the Germans and those in use in this country.

Figure 8 illustrates several operations on sections of the autobahnen just south of Stuttgart. The pictures were taken on July 23, 1936, and this particular section was inspected by the authors on July 28, 1947, just 11 years later. Figure 8*A* shows the mixer discharging a batch of concrete into the spreading hopper which then moves forward under the sunshade and discharges as shown in figure 8*B*. The spreader was moving from right to left when the photograph was taken. Note the very dry concrete, shown in greater detail in figure 8*D*. Figure 8*E* shows the Vögele finisher starting its second pass and figure 8*F* the final finish which, in this case, was only secured after several passes of the machine. Figure 8*C* is another view of the final surface. It also shows the woven-reed curing mats which were left in place for 3 weeks and were kept continuously wet during that period. This view also shows the light rail used to carry the sunshade frames. In his notes accompanying the photographs, Mr. McMillan states that the usual rate of progress, using machinery of the type described, was about 1,000 feet in 16 hours for full-width construction of 24 feet 7 inches.

Figure 9 illustrates operations on a section about 40 miles north of Kassel on which the Dingler machine was being used. Figure 9*A* shows the concrete mixer spanning the subgrade with the industrial railway on the right. Figure 9*B* shows the tamping machine in action compacting the lower course. Note again the dry consistency of the concrete. Also note that this particular operation was not protected by sunshades. Figure 9*D* illustrates the appearance of the concrete in the surface course before and after compaction by the tamping screed. Figure 9*E* shows the finished surface being given a light brooming and figure 9*C* shows the low sunshades used during the preliminary curing. Figure 9*F* is a general view of the pavement with the curing mats in place.

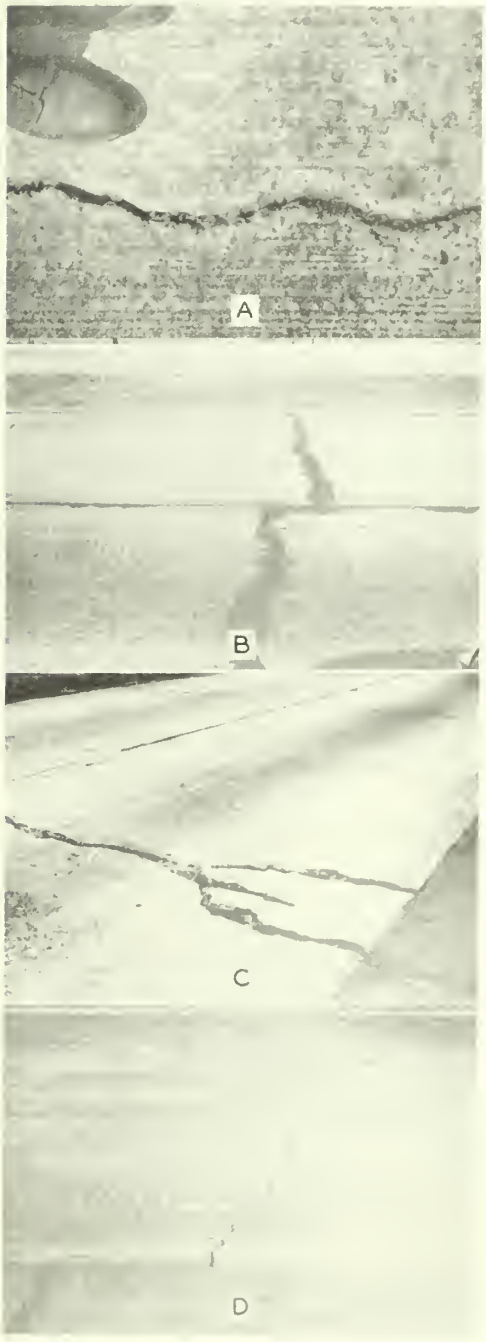

Figure 10.—Transverse cracking was common throughout the system.

PART III.—PRESENT CONDITION

Structural Defects

The structural defects that usually develop in concrete roads in the United States were found in the German pavements.

Transverse cracking was common and was not confined to any particular part of the system. There seemed to be no relation between transverse cracking and terrain or soil conditions. In most instances the cracks were closed and were not discernible at speeds normally traveled by automobiles. In some cases the cracks were open and the expansion joints had closed. This condition was noted on a section just north of Göttingen, near Kassel. Transverse cracking seems to be most pronounced on the route from Hannover to Frankfurt through the Ruhr Valley, from Frankfurt to Kassel, and from Frankfurt toward Heidelberg. The cracking shown in figure 10 is typical of that observed.

Spalling at transverse and longitudinal joints was found principally in the area around Frankfurt. At the transverse joints this appeared to be due in most instances to the infiltration of sand and other incompressible debris into the joint and the subsequent overstress of the concrete upon expansion of the pavement. Figure 11 illustrates the spalling at joints in the vicinity of Frankfurt.

Faulting at joints and at transverse cracks, in most instances less than ¼ inch, was observed in areas where there was evidence of a considerable amount of traffic. At a location approximately 50 miles west of Kamen faulting of joints in the outside lanes of the pavement was found to be ½ to ¾ inch. Faulting in the outside lane in the Ruhr-Frankfurt area is shown in figure 12*A*. The photograph was taken with the camera facing in the direction of the traffic and it should be noted that the expansion joints in the left lane, which had not faulted, are visible while those in the right lane had faulted sufficiently to be obscured.

In the München–Salzburg section there were a few small corner breaks. With few exceptions, these were the only corner breaks found in the entire mileage inspected. A

[9] *The mechanical finishing of concrete in road construction, with special reference to vibration. A new way of increasing output*, by R. Schade, 1940; translated from the German by L. Baumgartel; Library Communication No. 70, Department of Scientific and Industrial Research, British Road Research Laboratory, February 1947.

[10] Further details regarding construction equipment may be found in *Concrete road construction on the Reichsautobahnen*, by F. G. Turner, for the British Ministry of Transport. H. M. Stationery Office, London, 1939.

Figure 11.—Spalling at joints occurred principally in the vicinity of Frankfurt.

small corner break which has been patched is shown in figure 13*A*.

Some faulting and some spalling were observed at the longitudinal center joints. In many instances the longitudinal center joint was crooked. The longitudinal joints shown in figures 13*C* and *D* are typical of the condition frequently observed.

The riding surface of the pavement over a considerable portion of the mileage inspected was poor as compared with standards in the United States. For example, in the section from Berlin to Braunschweig the pavement was rough and there were many small areas where there was evidence that bush hammering had been resorted to in an effort to produce a better riding surface. On the other hand, there were many long stretches of smooth-riding pavement. Here again, there appeared to be no relation between the degree of smoothness and soil conditions, terrain, or other factors.

Mud Pumping and Warping

Evidence of mud pumping at joints in the concrete pavement was not found on the autobahnen. The absence of pumping over the areas where fine-grain soil subgrades predominate may be attributed to the passage of relatively few heavy loads and to the use of a layer of granular material such as sand or sand-gravel under the pavement. The sealing of the expansion joints and cracks to prevent the entrance of surface water was inadequate.

Warping was observed in a short section 31 miles south of Köln. The soil in this area is a silty clay and would be susceptible to sufficient volume change with an increase in moisture content from a dry compact state to cause warping. The layer of cohesionless granular material usually used under the pavement on the autobahnen would not be effective in preventing warping. Observation of this section was made during a light rainstorm and the water falling on the pavement was running away from the joints, indicating that they were high. This condition is illustrated in figure 12*B*.

The asphaltic surfaces of the shoulder strips were generally in good condition. Occasional local failures were noted.

Quality of the Concrete

So far as could be determined from an examination of the surface of the pavements and from chip samples taken from the corners or edges of the slabs, the quality of the concrete, with few exceptions, appeared to be excellent. The concrete had a good ring under the hammer and the difficulty encountered in breaking off the chip samples revealed, in general, a high degree of strength and toughness. As a further check on quality, a number of the chip samples were examined under the microscope in an effort to determine something of the character of the matrix, the presence and nature of voids, etc. Thin-section examinations were also made of many of the igneous aggregate types found in the samples. These data are summarized in table 2.

As will be noted, the examination of the chip samples revealed the concrete generally to be of excellent quality. There was no evidence of excess water. On the contrary, with the exception of sample No. 7, which was of bridge concrete, such voids as were noted were quite large and evidently of air origin, indicating that mixtures of very dry consistency had been used. This is borne out by the construction record to which reference has already been made. The bond between matrix and aggregate particles was in practically all cases sufficiently strong to fracture the aggregate particles themselves, again revealing good strength and probably a high degree of durability.

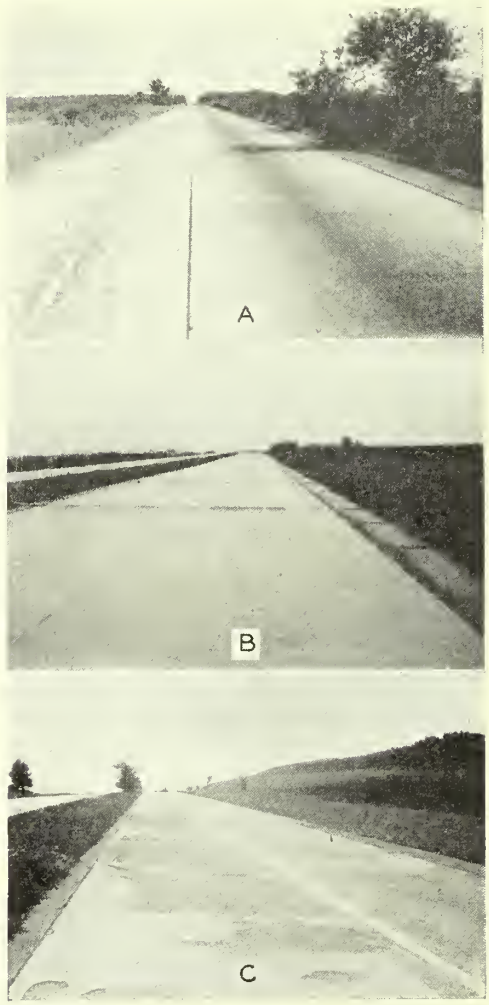

Figure 12.—(A) Joint faulting in the left lane (Ruhr-Frankfurt). (B) Warping resulted in high joints (Ruhr-Frankfurt). (C) Surface scale was seen only on the München-Salzburg route.

In the microscopic examination a magnification of 20 diameters was used, with a change to 50 diameters when some special detail seemed to indicate the desirability of the increased power. In none of the specimens was there any evidence of water gain beneath the lower surfaces of the aggregate particles.

Surface Texture

A characteristic feature of the pavement surface is the general absence of the rather heavy mortar top which we frequently leave on our roads and which so often scales off, leaving the surface rough and unsightly. An examination of the surface indicated, in most instances, the use of a mix that contained just enough paste to fill the voids in the aggregate and leave a small excess for finishing. Surfaces were dense and compact, with the coarse aggregate either just below the top or actually exposed in some degree. Typical illustrations of variations in the surface texture are shown in figure 14. Figure 14*A* is a view of a surface on which the screed marks are still showing. The corrugations are quite shallow and are about 1 inch apart. This particular photograph was taken just east of the Bad Oeynhausen interchange on the road to Hannover. Other sections showed the same condition, particularly on the inner lanes which, of course, carry less traffic. Figure 14*B* illustrates a fine-grained sand finish texture which was also noted in several places. The photograph was taken on the same road as figure 14*A* and about 5 miles east of the interchange.

Figures 14*D–F* illustrate varying degrees of coarse aggregate exposure. Figure 14*D* indicates the type of surface about 7 miles south of Frankfurt on the road to Stuttgart; figure 14*E* a section of the Hamburg–Lübeck road; and figure 14*F*, a section between Hannover and Berlin at kilometer post 194. Note in the last-mentioned illustration that coarse aggregate up to 1 inch in size, which is close to the maximum size used in the top course, is exposed. Many of the exposed aggregate surfaces are smooth, with a terrazzolike texture, probably the result of slight surface wear. In many places this type of surface was noted in the center of the traffic lanes (particularly the outside lane) with the original thin mortar surface showing at the extreme edges. In other sections where the coarse aggregate was exposed, particles of the hard igneous rock protruded slightly above the mortar, producing a slightly rough surface. This condition may have resulted from the scaling off of the light surface mortar or it may have been the result of wear—it is difficult to determine the exact cause. However, it should be emphasized that the slightly rough surface is not objectionable from the standpoint of ridability. Neither should the exposure of the aggregate have any detrimental effect on the ultimate durability of the concrete, since the quality of both aggregate and concrete was excellent.

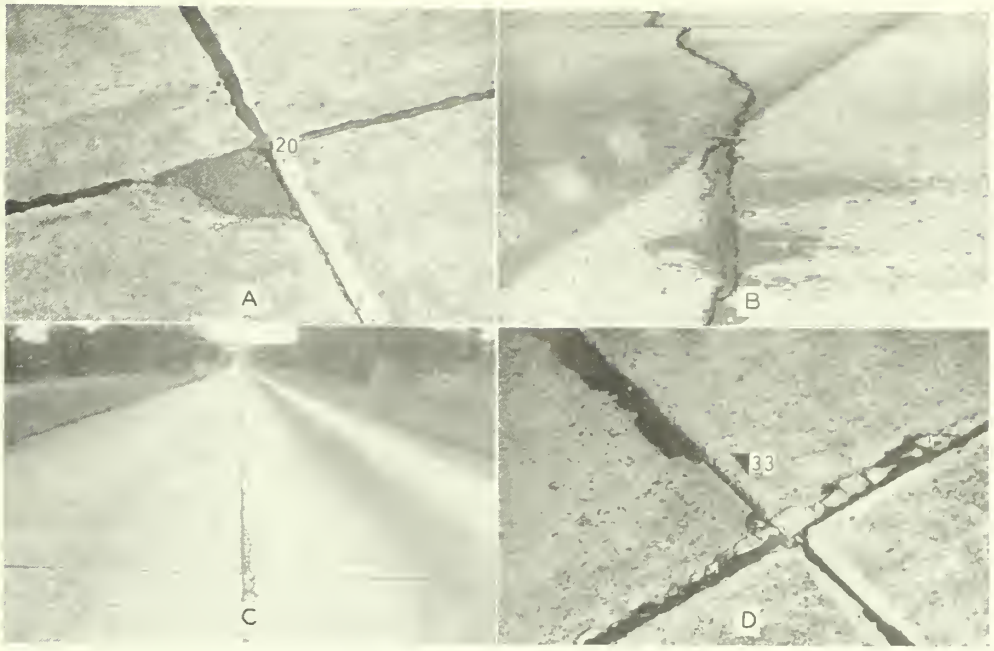

Figure 13.—(A) *A patched corner break (Frankfurt-Stuttgart).* (B) *A large random crack (Hannover-Ruhr).* (C, D) *Typical faulting of longitudinal joints (Ruhr-Frankfurt).*

There was evidence of surface grinding in several places. This was undertaken, evidently, to correct irregularities resulting from the difficulty of finishing the very dry concrete generally used.

A detail of the surface texture of black-colored concrete is shown in figure 14*C*.

Scale and Disintegration

With the exception of one section in southern Germany, the entire 1,000 miles of pavement covered by this inspection were found to be practically free from scale, either of the surface or of the progressive type. Only on certain stretches of the München–Salzburg section in Bavaria was surface scaling noted in any appreciable amount. Figure 12*C* is a general view of the pavement showing about the worst condition observed. This picture was taken

Table 2.—*Results of microscopic examination of chip samples*

Sample No.	Location	Aggregate types		Comment concerning—		
		Coarse	Fine	Aggregates	Matrix	Fracture
1	Hamburg–Bremen	Olivine basalt	Quartz, with sandstone, quartzite, and chalcedony.	Sound. No weathered particles.	Mass crystalline. Excellent bond on aggregate surfaces.	Through aggregate particles.
2	Frankfurt-Stuttgart, 20 miles south of Frankfurt.	Basalt	Quartz, quartzite with some chalcedony and limestone.	do	Mass crystalline and well bonded to aggregate.	Do.
3	Frankfurt-Stuttgart, 34 miles south of Heidelberg.	Rhyolite	Quartz, feldspar, and chalcedony.	do	Mass crystalline and well bonded to aggregate. Excellent bond. No limy deposit in air pockets.	Do.
4	Stuttgart-Münehen, 15 miles south of Stuttgart.	Limestone and syenite.	Quartz, limestone, chalcedonic chert, granite, feldspar.	Sound and strong. Break surfaces of yellowish tint. Possibly some weathering.	Mass crystalline and well bonded to aggregate. No limy deposit in air voids.	Through aggregates.
5	München-Salzburg, 20 miles west of Austrian border.	Altered granite	Sandstone, quartz, granite, limestone, feldspar.	Sound	Mass crystalline. Mortar very dense.	Through nearly all aggregate particles.
6	Hannover-Ruhr, 30 miles west of Bad Oeynhausen.	Basalt	Quartz, quartzite, plus other unknown fragments.	Souud, unweathered materials.	Mass crystalline	Through aggregate particles.
7	Ruhr-Frankfurt (bridge)	Basalt, feldspathic sandstone.	Quartz, syenite, feldspar, feldspathic sandstone.	Sound. No evidence of weathering.	Mass crystalline. Water voids show no evidence of limy deposit. Strong and dense in appearance.	
8	Ruhr-Frankfurt, 28 miles south of Leichingen.	Basalt	Quartz, granite, feldspar	Sound. One weathered particle.	Mass crystalline with few air voids. No evidence of limy deposit. Dense, strong concrete.	
9	Frankfurt-Stuttgart, 20 miles south of Frankfurt.	Granodlorite, feldspathic sandstone.	Quartz, quartzite, feldspar	Sound	Light gray; mass crystalline. Few air voids. No limy deposit.	Few fractured aggregate particles, indicating lower bond strength than in other samples.

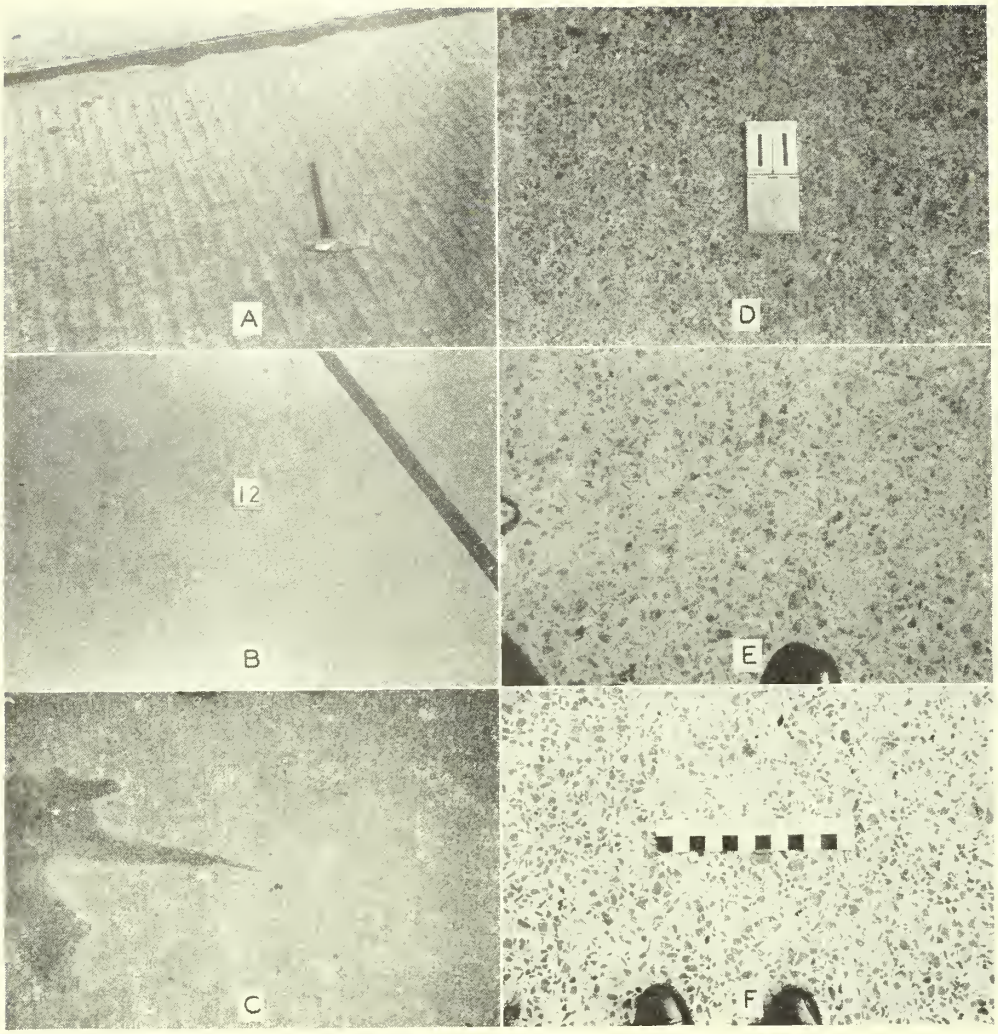

Figure 14.—(A) In some places screed marks still appear as shallow corrugations. (B) A fine-grained sand finish. (C) Black colored concrete. (D–F) Typical appearance of varying degrees of coarse aggregate exposure.

about 20 miles west of the Austrian border at Salzburg. Figures 15*A* and *C* are details showing the type of scaling that has developed on this section. It is apparently of the surface type only; that is, the concrete below the scaled areas appeared to be sound. Figures 15*A* and *C* indicate a rather heavy layer of surface mortar at this point and this fact, coupled with the possibility that the winters in this region, due to its high elevation, may be somewhat more severe than elsewhere in western Germany, may account for the scaling. However, in this connection it should be noted that, according to weather records as shown in table 1, the average January temperature at München was only slightly lower and the annual rainfall only slightly higher than for the more northern cities.

It has been stated that the distress which developed on the München–Salzburg section at a relatively early period was due to the use of inferior aggregate. A chip sample (No. 5 in table 2) taken at the approximate location shown in figure 15*A* did not, however, reveal the presence of an appreciable amount of inferior material, although the petrographer did indicate that the coarse aggregate in this sample was an "altered granite." Just how much the generally poor subgrade conditions in this area, or other factors, may have contributed to the scaling is not known.

Only isolated scaled areas, all of the surface type, were noted elsewhere on the system. Figure 15*D* is a detail of a local scaled area on the Frankfurt–Stuttgart section, a few miles south of Frankfurt. From the standpoint of general condition this is one of the poorest sections on the autobahnen. As previously noted, considerable spalling at joints (see fig. 11) has also developed on this section. Figure 15*B* is a detail showing a thin surface scale on the Hannover–Ruhr section, about 30 miles east of the Bad Oeynhausen interchange. This pavement, as well as many other portions of the system, had evidently been sprayed with bituminous material at some time, probably for camouflage purposes. The scale here is not deep enough to affect the riding quality of the surface.

Incipient disintegration of the concrete, as revealed by the formation of so-called D lines along the edges of the pavement and along joints, was found at only one place on the system; a point on the München–Salzburg section where the worst scaling was found. The D lines in this case were found along the outside (north) lane and were very fine. The condition is shown in figures 15*E* and *F*. In the United States, cracks of this type have come to be associated with the beginning of disintegration due to accelerated weathering.

The general freedom of the motor roads from scaling and other troubles associated with severe weathering may be attributed to the following factors, the listing not necessarily being in order of importance:

1. The general excellent quality of the aggregates.
2. The low water-cement ratio (usually 0.45 or less, by weight).
3. The practice of designing mixtures and compacting concrete in such a manner as to avoid the formation of a heavy surface layer of mortar.
4. The comparatively mild winters.
5. The fact that chloride salts were not used for ice removal.
6. Thorough curing.

The possible influence of all of these factors is fairly obvious. The role of the cement is not so apparent. According to Swayze's report, the cements used in the autobahnen were distinctly inferior to those manufactured in this country. However, the concrete is good and from this fact we can conclude at least that it is possible to make good concrete with the German cements provided the other conditions are right. Whether these cements would perform any better than the American cements under conditions that prevail in the northern part of the United States is another matter. An intensive research program would be necessary to answer this question.

PART IV.—SUMMARY AND RECOMMENDATIONS

As mentioned in the body of this report, the comparatively light traffic on the German motor roads, particularly as regards the movement of heavily loaded vehicles, as well as the comparatively mild climate, make comparisons with the performance of concrete pavements in this country difficult. This is especially true if we attempt to compare the German primary roads with the heavy-duty concrete pavements of the industrial States of the north and middle west. Many of these roads undoubtedly carry a far heavier volume of truck traffic than any of the motor roads in Germany. The question therefore is: How would the German roads perform under similar climatic and traffic conditions? It is, of course, impossible to say definitely. However, it is the authors' opinion that these two factors—the low volume of heavy truck traffic and the relatively mild climate—probably account for the comparative freedom of the German motor roads from structural defects and that, under comparable climatic and traffic conditions, the structural performance of the German roads would be about the same as the average concrete pavement in this country. This opinion is borne out by the fact that structural failures such as joint spalling, faulting, etc., were found to be more numerous and, in general, more serious on sections of the system where the indications pointed to the heaviest traffic.

In this opinion, the authors also recognize the beneficial effect of using a layer of granular material under the pavement, particularly with respect to the control of mud pumping at joints, a condition which was not found anywhere on the system.

The quality of the German concrete was excellent. Although the design strengths were not unusually high as compared to standards in use in the United States, the indications were that a high degree of uniformity was obtained. This may possibly be due to the care used in proportioning mixes, as, for example, the use of several sizes of aggregate and the practice of limiting the maximum size in the top course to about 1 inch. Reference was also made to the general absence of the relatively heavy mortar top which we see so often on our roads, and to the fact that this might have reduced the tendency to scale. It is believed that the principle of consolidating the pavement slab by vibration is good although, in the authors' opinion, the Germans may have gone too far in their effort to use excessively dry concrete. This is indicated by the rather rough surface with occasionally noted evidence of grinding off of high spots. However, the principle of using as dry a mix and as low-sanded a mix as can be consolidated properly by the effective use of vibratory equipment is good and, in the authors' opinion, should be encouraged in this country.

The uniformly excellent durability of the concrete is probably also due, in part at least, to the excellent quality of the aggregates generally used. This is also a factor to which we might give more consideration in the United States.

The probable influence of long continued and thorough curing also should not be ignored. Whether the elaborate precautions for protecting the fresh concrete from sun or wind were always necessary is problematical. However, the practice of thorough water curing, although old-fashioned in terms of modern American practice, is still, in the opinion of the authors, the preferred method.

The survey revealed nothing regarding cement except that, regardless of the quality of the cement used, the quality of the concrete was excellent; and also the fact that concrete containing Eisenportland (portland–blast-furnace slag cement) was, on the average, as good as concrete containing straight portland cement. Although it was impossible to determine on just what portions of the autobahnen the Eisenportland was used, it was stated to have been used on about one-third of the total mileage. This fact would seem to warrant the statement that its performance was satisfactory.

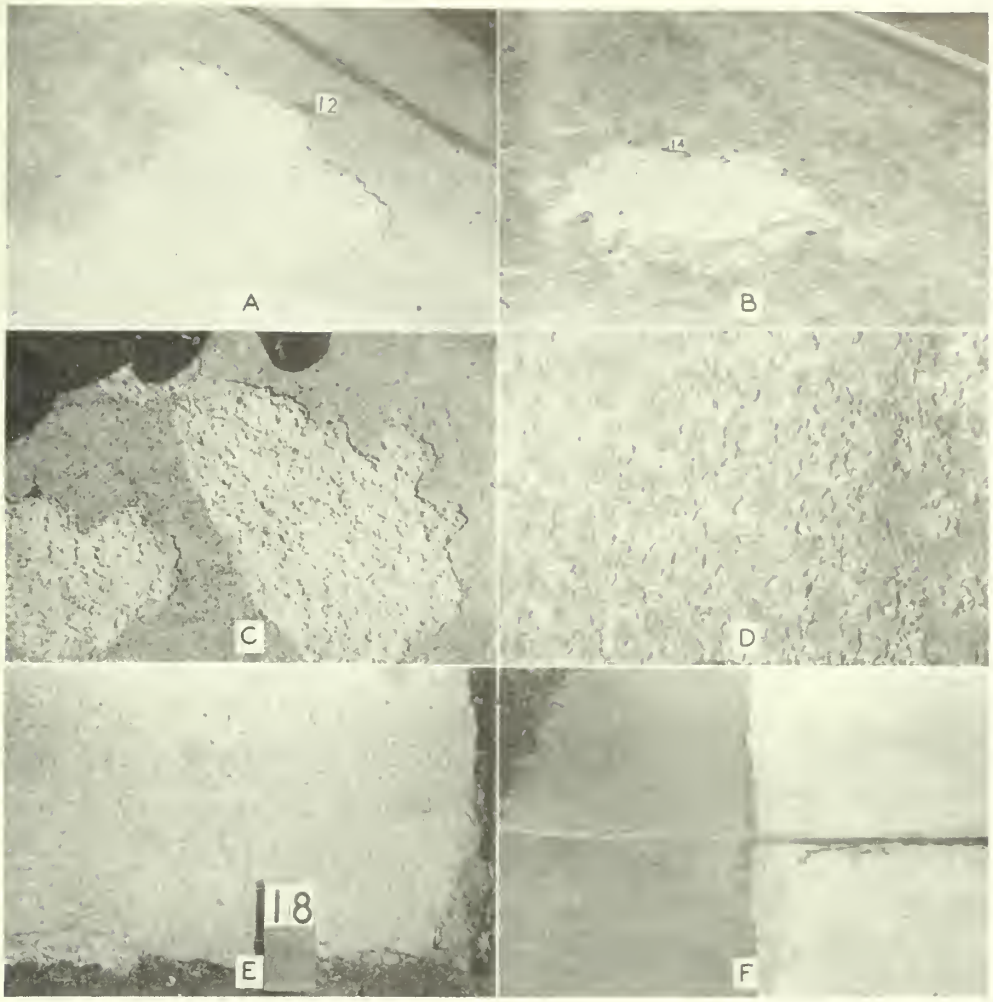

Figure 15.—(A–D) Surface scale occurred commonly on the München-Salzburg route but only isolated cases were observed elsewhere. (E, F) Incipient disintegration, revealed by D lines, was found only at one place on the system.

Recommendations

As the result of their examination of concrete pavements on the German motor-road system the authors recommend that steps be taken to initiate a comprehensive program of research on each of the following subjects:

1. Study of the possibility of insuring greater uniformity in pavement concrete by reducing the maximum size of the coarse aggregate.

2. Development of more effective methods of compacting concrete in pavements by mechanical means, such as vibration, tamping, etc.

3. Study of the effects of variations in the chemical composition of cements and the methods of manufacturing cements on the properties of concrete. In this work the authors would go considerably outside the range in composition and fineness now being studied under the program of the "Long-time study of cement performance in concrete." [11] Work of this nature should be carried out by the manufacturers and might well be accomplished by an extension of the present program of the long-time study to cover these variables.

[11] For a description of this project, see *Progress in the long-time study of cement performance in concrete*, by F. R. McMillan; Proceedings of the American Concrete Institute, vol. 13, No. 5, April 1942.

Analysis of Rectangular Reinforced Concrete Sections Subjected to Direct Stress and Bending in Two Directions

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The problem of determining the unit stresses in a reinforced concrete section subjected to direct stress and bending in two directions often occurs in the analysis of continuous frames, and a solution that simplifies the work and removes some of the drudgery is indeed welcome. The method presented in this article is an improvement over procedures now being used since it eliminates one of the variables—the slope of the neutral axis of the section—at an early stage. The slope is determined within a close degree of correctness, with fairly simple calculations, by first considering the entire area of the cross section as effective.

ALTHOUGH a wealth of material has been presented to the engineering profession on the subject of direct stress and bending in reinforced concrete sections, most of the data presently available are limited to the special case of the general problem; i. e., direct stress plus bending in only one direction. The more general problem, direct stress plus bending in two directions along axes 90 degrees apart, has been treated in general terms only by Cross and Morgan,[1] [2] and more specifically by Saville.[3] The standard specifications of the American Association of State Highway Officials [4] gives the general equation for determining the position of the neutral axis for the general case, but this equation is not very helpful without a knowledge of how to apply it.

The problem of determining the unit stresses in a reinforced concrete section subjected to bending and direct stress in two directions occurs rather frequently in practice. Everyday examples of this problem, among many others, are building columns in ordinary beam-and-girder construction with bending induced by girders in two directions framing into the columns, open-framed bridge bents and abutments in continuous construction with bending induced by longitudinal girders and transverse cross beams, and the single column or umbrella-type bridge pier with bending induced by longitudinal girders and the transverse cross beam at the top of the pier. While the analysis of members of this type is not particularly difficult, it is rather long and involved. The purpose of this article is to demonstrate the method outlined by Cross by applying it to a specific example, and to show the general applicability of the method. Suggestions for helping the designer to make more reasonable assumptions in the first trial and thus reduce the number of solutions required to bring the assumed and actual positions of the neutral axis to sufficiently close agreement are also given. The reader is referred to Cross's works for the derivation of the formulae employed.

NOTATION

The notation used in this article is for the most part standard and is as follows:

a is the area, in square inches, of a portion of the section. In the case of the steel reinforcing, it is the transformed area of the steel in terms of the concrete.

A is the total effective area of the section, including the transformed area of all reinforcement.

X and Y axes are the coordinate axes chosen to compute the properties of the section. These axes may pass through any desired point, but should be chosen so as to simplify the calculations of the section properties. In general, these X and Y axes are not through the center of gravity of the effective cross section.

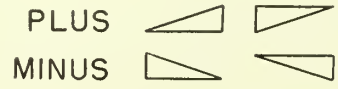
x and y are the coordinates of any point referred to the X and Y axes arbitrarily chosen to compute the section properties, as defined above.

ax and ay are the products of the elementary areas and the coordinates of the center of gravity of those areas referred to the X and Y axes. These products are really the statical moments of the elementary areas about the X and Y axes.

$I_{yy}=ax^2+i_{yy}$. This value is the moment of inertia in inches^4 of the effective area of the cross section (including the transformed area of the reinforcement) about the Y axis. It is computed by summing the moments of inertia of the several elementary areas obtained by the familiar transfer axis theorem, i. e., the product of the elementary area and the coordinate x to its center of gravity squared, plus the moment of inertia of the area about the y axis through its own centroid.

$I_{xx}=ay^2+i_{xx}$. This value is the moment of inertia in inches^4 of the effective area of the cross section (including the transformed area of the reinforcement) about the X axis. It is computed by summing the moments of inertia of the several elementary areas obtained by the transfer axis theorem, i. e., the product of the elementary area and the coordinate y to its center of gravity squared, plus the moment of inertia of the area about the x axis through its own centroid.

$I_{xy}=axy+i_{xy}$. This value is the product of inertia in inches^4 of the effective area of the cross section (including the transformed area of the reinforcement) about the X and Y axes. It is computed by summing the products of inertia of the several elementary areas obtained by multiplying the elementary area by the coordinates x and y of its center of gravity, plus the product of inertia of the elementary area about the x and y axes through its own centroid. In dealing with rectangular sections, or irregular sections bounded by straight lines, the elementary area will be either a rectangle or a triangle or a combination of both. For any rectangular section, $i_{xy}=0$. For any triangular area about x and y axes through its center of gravity and parallel to the base and altitude of the triangle, $i_{xy}=\frac{b^2h^2}{72}$, where b and h represent the base and altitude, respectively, of the triangle. The product of inertia of a triangle will be positive in sign if its hypotenuse slopes downward to the left and negative in sign if its hypotenuse slopes downward to the right, as indicated in the sketch below.



[1] *The column analogy*, by Hardy Cross; University of Illinois Engineering Experiment Station Bulletin No. 215; October 1930.

[2] *Continuous frames of reinforced concrete*, by Hardy Cross and N. D. Morgan; John Wiley & Sons; 1932.

[3] *Analyzing non-homogeneous sections subjected to bending and direct stress*, by W. G. S. Saville; Civil Engineering, vol. 10, No. 3, March 1940; p. 170.

[4] *Standard specifications for highway bridges (1944)*; American Association of State Highway Officials.

$M_{vv}=Px$ and $M_{xx}=Py$. These values are the applied moments, in inch kips, about the X and Y axes. They are computed as the product of the direct load P and the coordinate x or y of its point of application.

X_o and Y_o axes are the coordinate axes parallel to the X and Y axes first chosen to compute the properties of the section but passing through the center of gravity of the total effective area A.

x_o, y_o, $I_{x_ox_o}$, $I_{v_ov_o}$, $M_{x_ox_o}$, $M_{v_ov_o}$, $I_{x_ov_o}$. These values are the section properties and the applied moments referred to the X_o and Y_o axes.

The following values are the section properties about the X_o and Y_o axes through the center of gravity of the effective area A but corrected for the dissymmetry of the section:

$$I'_v=I_{v_ov_o}-I_{x_ov_o}\frac{I_{x_ov_o}}{I_{x_ox_o}}$$

$$I'_x=I_{x_ox_o}-I_{x_oy_o}\frac{I_{x_ov_o}}{I_{v_ov_o}}$$

$$M'_v=M_{v_ov_o}-M_{x_ox_o}\frac{I_{x_ov_o}}{I_{x_ox_o}}$$

$$M'_x=M_{x_ox_o}-M_{v_ov_o}\frac{I_{x_ov_o}}{I_{v_ov_o}}$$

The equation that follows is that of the neutral axis or line of zero stress referred to axes X_o and Y_o, and coordinates x and y in this equation are also measured from axes X_o and Y_o:

$$\frac{P}{A}+\frac{M'_v}{I'_v}x+\frac{M'_x}{I'_x}y=0.$$

X_n and Y_n are the distances from the neutral axis to the extreme fibers of the section, measured parallel to the X_o and Y_o axes.

f is the fiber stress at any point and is equal to

$$\frac{M'_x}{I'_x}Y_n \text{ or } \frac{M'_v}{I'_v}X_n$$

f_c and f_s are the unit stresses, in pounds per square inch, in the concrete and steel, respectively.

METHOD OF ANALYSIS

To compute the unit stresses in a rectangular reinforced concrete section subjected to bending moments in two directions and direct stress the designer should proceed as follows (it is assumed that a trial section complete with reinforcement has been tentatively selected for analysis and this section drawn accurately to scale):

1. Estimate the position of the neutral axis of the section. This estimate of the position of the neutral axis requires judgment and experience since it involves drawing a line on the sketch of the section with position and slope unknown. Moreover, since the succeeding calculations are rather lengthy, an inordinate amount of time may be spent in obtaining sufficiently close agreement between the assumed and the computed positions of the neutral axis unless the position first selected is approximately correct. It has been found that one variable, the slope of the actual neutral axis, may be determined quite accurately by first considering the entire area of the cross section as effective (which is equivalent to assuming that the concrete takes its full share of the tension induced by the bending moments) and computing the position of the neutral axis on this basis. The slope of the final neutral axis will be found to deviate only slightly from the slope of this fictitious neutral axis.

It is recommended, then, that the first step in the solution consist of computing the location of this fictitious neutral axis, considering the entire transformed area of the section effective, and plotting it upon the sketch. The first trial neutral axis for the cracked section should then be drawn parallel to this fictitious neutral axis and on the compression side of the section from it. Thus one variable has been eliminated. Also, if the stresses on the tension side of the section when computed considering the concrete to take tension are moderate, say 150 p. s. i. or less, it will be unnecessary to proceed with the more exact calculation. The remaining variable, the distance from the fictitious neutral axis of the entire section to the assumed trial neutral axis of the cracked section, will depend upon the eccentricity of the direct load, and must be estimated based upon the amount of this eccentricity.

2. The trial neutral axis of the cracked section having been selected, the designer proceeds to the calculation of the section properties about some convenient pair of axes. These quantities should preferably be tabulated, using some form similar to that shown later on in this article. After all of the section properties have been computed, tabulated, and totaled, they must then, in general, be corrected for two factors: First, they must be reduced to the X_o and Y_o axes passing through the centroid of the cracked section and parallel to the arbitrarily chosen X and Y axes used for computation purposes; and second, they must be corrected for the dissymmetry of the section.

3. The position of the centroid of the cracked section is found by dividing the sum of the statical moments of the elementary areas about the X and Y axes by the area A, the sum of the elementary areas. Thus $\sum\frac{ax}{A}$ and $\sum\frac{ay}{A}$ will give x_c and y_c, the coordinates of the centroid of the cracked section measured from the X and Y axes. The corrections to the quantities I_{vv}, I_{xx}, and I_{xv} are respectively Ax_c^2, Ay_c^2, and Ax_cy_c, and are always subtracted algebraically from the summations about the axes X and Y. Likewise, the corrections to be applied to M_{vv} and M_{xx} are, respectively, Px_c and Py_c and are also always subtracted algebraically. The values after these corrections have been made represent the section properties and applied moments about the X_o and Y_o axes and are the quantities $I_{v_ov_o}$, $I_{x_ox_o}$, $I_{x_ov_o}$, $M_{v_ov_o}$, and $M_{x_ox_o}$.

4. The second and last series of corrections to be applied to the quantities enumerated in the last sentence of the preceding paragraph are the corrections for the dissymmetry of the section. The corrections to be applied to $I_{v_ov_o}$ and $I_{x_ox_o}$ are

$$I_{x_ov_o}\frac{I_{x_ov_o}}{I_{x_ox_o}} \text{ and } I_{x_ov_o}\frac{I_{x_ov_o}}{I_{v_ov_o}},$$

respectively, and are always subtracted algebraically from $I_{v_ov_o}$ and $I_{x_ox_o}$, giving the final quantities I'_v and I'_x. The corrections to be applied to $M_{v_ov_o}$ and $M_{x_ox_o}$ are

$$I_{x_ov_o}\frac{M_{x_ox_o}}{I_{x_ox_o}} \text{ and } I_{x_ov_o}\frac{M_{v_ov_o}}{I_{v_ov_o}},$$

respectively, and are subtracted algebraically from $M_{v_ov_o}$ and $M_{x_ox_o}$, giving the final quantities M'_v and M'_x.

5. The equation of the neutral axis is then

$$\frac{P}{A}+\frac{M'_v}{I'_v}x+\frac{M'_x}{I'_x}y=0.$$

After substituting the numerical values in this equation and simplifying, the intercepts of the neutral axis on the sides of the section are obtained by placing values of x and y, the coordinates of the sides of the section measured from the X_o and Y_o axes in the equation, and computing the resulting values. These values are then compared with the actual values of the assumed position of the neutral axis. If the agreement is reasonably close, say within approximately 1 inch, no further trials will be necessary. If, however, the computed and actual intercepts of the neutral axis on the sides of the section differ by more than about 1 inch, then a new trial neutral axis should be selected and the entire process thus far described repeated.

6. When the designer has a solution wherein the computed and assumed positions of the neutral axis agree within the limits of error desired, all the data will be available for computing the actual stresses in the steel and concrete. The general equation for the stress f is:

$$f=\frac{M'_x}{I'_x}Y_n \text{ or } f=\frac{M'_v}{I'_v}X_n.$$

If the sketch has been accurately drawn to scale, the values of Y_n or X_n may be scaled. If computed values are desired, they may be found by solving the geometric figures composing the sketch, or they may be computed by substituting the proper values into the general equation for the neutral axis. X_n and Y_n may be the distances, measured parallel to the X_o and Y_o axes, to any point where the stresses are desired. However, the maximum stress values for the concrete and steel are usually the only values in which the designer is interested. Accordingly, the maximum distances X_n and Y_n, when substituted in the equations for stress given above, will give the maximum stresses desired. The stress in the steel is found, of course, by multiplying the stress computed from the equation at the beginning of this paragraph by the value of n for the concrete strength being used.

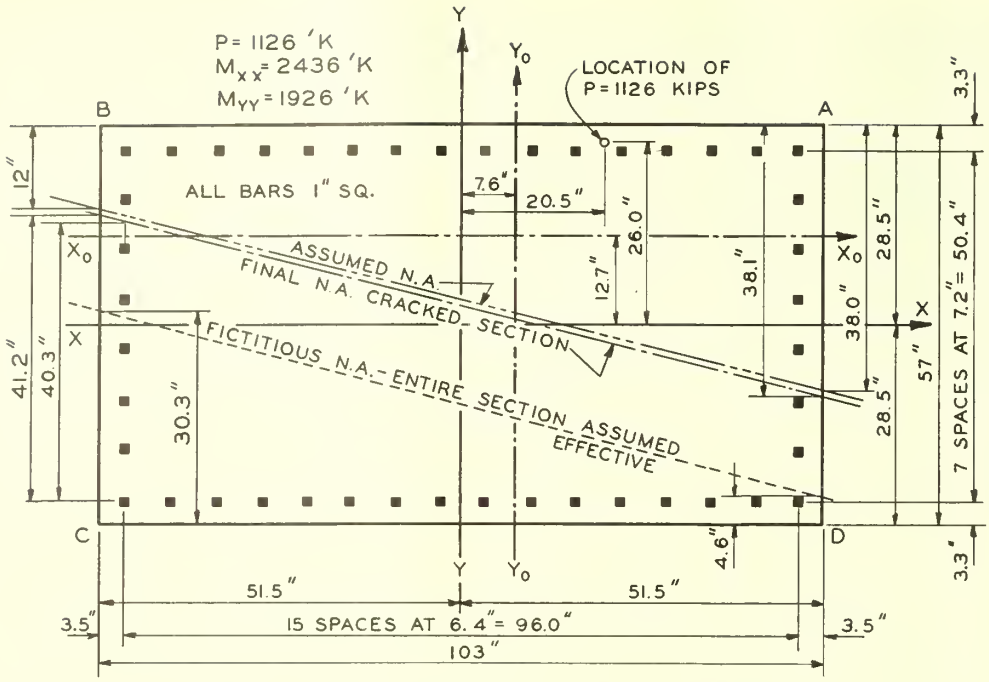


Figure 1.—Scale drawing of section analyzed in example.

APPLICATION TO AN EXAMPLE

In illustration of the analysis process, a specific example has been assumed and followed through, step by step.

Step 1. Make a scale drawing of the section for which the stresses are to be computed, as shown in figure 1. The assumed X and Y axes are included in the drawing. The three neutral axes shown in figure 1 are not drawn as part of the original sketch, nor are axes X_o and Y_o, since their positions are determined by the calculations that follow.

Step 2. The values of P, M_{xx}, and M_{yy}, shown in figure 1, are known. From these values the distances x and y from the X and Y axes to the point of application of the load are computed as follows:

$$x=\frac{M_{yy}}{P}=\frac{1,926}{1,126}\times 12=20.5 \text{ inches.}$$

$$y=\frac{M_{xx}}{P}=\frac{2,436}{1,126}\times 12=26.0 \text{ inches.}$$

Step 3. Compute I_{xx} about the X axis which was chosen through the centroid of the total area:

$$\text{Concrete}=\frac{bh^3}{12}=\frac{103\times 57^3}{12}=1,589,600.$$

Steel (44 1-inch squares) = summation of ay^2 (the transformed area of each bar = nA_s, and for $n=10$, each bar is equivalent to $10\times 1=10$ square inches of concrete):

$$\begin{aligned} 32\times 10\times 25.2^2&=203,200\\ 4\times 10\times 18.0^2&=12,900\\ 4\times 10\times 10.8^2&=4,700\\ 4\times 10\times 3.6^2&=500\\ \text{Total}&=221,300 \end{aligned}$$

Total $I_{xx}=1,589,600+221,300=1,810,900$.

Step 4. Compute I_{yy} about the Y axis which was chosen through the centroid of the total area:

$$\text{Concrete}=\frac{57\times 103^3}{12}=5,190,400.$$

Steel = summation of ay^2, with $a=10$ for each bar:

$$\begin{aligned} 16\times 10\times 48^2&=368,600\\ 4\times 10\times 41.6^2&=69,200\\ 4\times 10\times 35.2^2&=49,600\\ 4\times 10\times 28.8^2&=33,200\\ 4\times 10\times 22.4^2&=20,100\\ 4\times 10\times 16.0^2&=10,200\\ 4\times 10\times 9.6^2&=3,700\\ 4\times 10\times 3.2^2&=400\\ \text{Total}&=555,000 \end{aligned}$$

Total $I_{yy}=5,190,400+555,000=5,745,400$.

Step 5. Compute total area:
$A=103\times 57+44\times 10=6,311$ square inches.

Step 6. Compute

$$f=\frac{P}{A}\pm\frac{M_{yy}}{I_{yy}}x\pm\frac{M_{xx}}{I_{xx}}y$$

$$=\frac{1,126,000}{6,311}\pm\frac{1,926,000\times 12}{5,745,400}\times$$

$$51.5\pm\frac{2,436,000\times 12}{1,810,900}\times 28.5$$

$$=178\pm 207\pm 460.$$

Then:

f at point $A=+845$;
f at point $B=+431$;
f at point $C=-489$;
f at point $D=-75$.

Step 7. Compute the location of the neutral axis:

Along side DA from point $D=\frac{75}{920}\times 57=$ 4.6 inches.

Along side CB from point $C=\frac{489}{920}\times 57=$ 30.3 inches.

These calculations locate the fictitious neutral axis with the entire section assumed effective. This fictitious neutral axis is drawn on the sketch, as shown in figure 1.

Step 8. An assumed neutral axis for the cracked section is now selected and drawn on the sketch (see fig. 1), parallel to the fictitious neutral axis and on the compression side of the section from it. The section properties of the trial neutral axis as related to the X and Y axes are computed, tabulated, and totaled as shown in the first four lines of figure 2. The entries in the first line are for the steel reinforcement, while those in the second and third lines are for the concrete, considered as a rectangle and a triangle which, combined, include the total area of the cracked section.

It will be observed that ax and ay for the steel are zero because of the symmetry of the section. Values of I_{yy} and I_{xx} for the steel are the same as those computed in steps 3 and 4 for the uncracked section. The values i_{xy} and axy for the steel are zero because of the symmetry of the reinforcement. Values of i_{yy} and i_{xx} for the triangular section of the concrete are moments of inertia of the triangle about its own centroidal axes, and are $\frac{bh^3}{36}$ and $\frac{hb^3}{36}$. The value of i_{xy} for the triangle is computed as $\frac{b^2h^2}{72}$, with the sign determined as previously explained.

Step 9. The computed properties are corrected to the centroid in the fifth line of the tabulation (fig. 2). The values of x_c and y_c are computed as $\sum\frac{ax}{A}$ and $\sum\frac{ay}{A}$, respectively. The X_o and Y_o axes can now be drawn on the sketch (fig. 1). The corrections are computed as previously explained, and are subtracted algebraically from the totals in the line above.

Step 10. In the last line of the tabulation (fig. 2), corrections for the dissymmetry of the section are made. These are computed as previously explained, and are subtracted algebraically from the figures in the line above.

Step 11. From the values computed in the tabulation, the equation of the neutral axis, previously given, may be solved as follows:

$$\frac{1,126,000}{3,015}+\frac{20,760,000}{2,567,500}x+\frac{16,090,000}{491,000}y=0;$$

$$373+8.09x+32.7y=0;$$

$$y+0.248x=-11.4.$$

For the assumed trial neutral axis (fig. 1), x equals $+43.9$ or -59.1 and y equals -22.2 or $+3.8$. Substituting the values of x in the equation above,

if $x=+43.9$, $y=-22.3$;

if $x=-59.1$, $y=+3.3$.

The computed values of y are in close agreement with the actual values of the assumed position of the neutral axis, and no further trial is necessary.

Part of Section	Area a (sq. in.)	x (in.)	y (in.)	ax	ay	$I_{yy} = ax^2 + i_{yy}$		$I_{xx} = ay^2 + i_{xx}$		$I_{xy} = axy + i_{xy}$		$M_{yy} = Px$ (in. kips)	$M_{xx} = Py$
Steel (44 1-in.sq.,n=10)	440	varies	varies	0	0	555,000	0	221,300	0	0	0	23,110	+29,230
Concrete 103" × 12"	1,236	0	+22.5	0	+27,810	0	1,092,700	625,700	14,800	0	0	—	—
Concrete 103" × 26"	1,339	+17.17	+7.83	+22,990	+10,490	394,600	789,200	82,200	50,300	+179,900	-99,600	—	—
Totals	$A =$ 3,015	—	—	+22,990	+38,300	949,600	+1,881,900	929,200	+ 65,100	+179,900	-99,600		
						2,831,500		994,300		+80,300		+23,110	+ 29,230
Correct to centroid		$x_c =$ +7.63	$y_c =$ +12.70	Correction		175,500		486,300		+292,300		+8,590	+14,300
				Corrected values		2,656,000 $= I_{y_0}$		508,000 $= I_{x_0}$		−212,000 $= I_{x_0 y_0}$		+14,520 $= M_{y_0} y_0$	+14,930 $= M_{x_0} x_0$
Correct for dissymetry of the section				Correction		88,500		17,000		—		-6,240	-1,160
				Corrected values		2,567,500 $= I_y'$		491,000 $= I_x'$		—		+20,760 $= M_y'$	+16,090 $= M_x'$

Figure 2.—Tabulation for analysis of cracked section.

Step 12. The actual stresses may now be computed from the equation previously given:

$$\text{Maximum } f_c = \frac{+16,090,000}{491,000} \times 38.1$$

$$= +1,250 \text{ p. s. i. (compression);}$$

$$\text{Maximum } f_s = \frac{+16,090,000}{491,000} \times -40.3 \times 10$$

$$= -13,200 \text{ p. s. i. (tension).}$$

These results may be checked by computing the intercepts of the neutral axis on the top and bottom of the section. The method of computation has been suggested previously. Using the values so calculated:

$$f_c = \frac{+20,760,000}{2,567,500} \times +153.5$$

$$= +1,240 \text{ p. s. i. (compression);}$$

$$f_s = \frac{+20,760,000}{2,567,500} \times -162.4 \times 10$$

$$= -13,150 \text{ p. s. i. (tension).}$$

These agree very closely with the values computed above.

The discussion and calculations in this article are based on inch and pound units, but any other consistent set of units could be used. Theoretically, the area of the reinforcement embedded in the concrete on the compression side of the section should be multiplied by $(n-1)$ instead of n in computing its transformed properties, but this refinement is considered of negligible importance.

The designer who will study the explanations and example that have been presented should have no difficulty in applying the general method to his own particular problems. As suggested by Saville, this method may also be applied to a variety of problems, including the computation of foundation pressures when the foundation is composed of materials having widely different supporting powers, and when overturning moments are applied in more than one direction.

U. S. GOVERNMENT PRINTING OFFICE 1948

The Effect of Characteristics of Asphalts on Physical Properties of Bituminous Mixtures

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by R. H. LEWIS, Senior Chemist, and J. Y. WELBORN, Materials Engineer

Laboratory tests on a particular bituminous mixture and a review of other investigations of the physical properties of bituminous mixtures indicate that these physical properties can be correlated, before and after accelerated or natural weathering, with the characteristics of the contained binder. Changes occurring in the binders of asphaltic mixtures that influence the durability of pavements under service conditions can be measured directly by tests on the bitumen recovered from the mixtures, or indirectly by the changes in the physical properties of the mixtures.

IN RECENT years many investigations have been made dealing with the durability of various types of pavements in which asphaltic materials have been used as binders. Two general methods of attack have been followed in these investigations:

1. Study of the physical properties of asphaltic mixtures and correlation of these properties with durability. Compressive strength, flexural strength, shear resistance, impact, Hubbard–Field stability, abrasion, and other tests have been used by various investigators as a means of studying the physical properties of mixtures. Some of the studies were made on representative samples taken from pavements, their physical properties determined by one or more of the above tests, and the results correlated with pavement condition. Other studies have been made on laboratory-prepared mixtures and their physical properties determined before and after subjecting to various accelerated weathering tests.

2. Study of the characteristics of asphaltic binders before and after exposure to laboratory oxidation tests or to actual service conditions and correlation of the test results with pavement durability. Some investigators have studied the durability problem by recovering the asphaltic binders from pavements and correlating the charaeteristics of the asphalt with service behavior. Other studies have been made by determining the effect of laboratory oxidation tests on the asphaltic material itself and comparing the characteristics of the residues with those of the original material.

A review of published literature on the study of the durability of asphaltic materials reveals that there has been little attempt to correlate the physical properties of bituminous mixtures with the characteristics of the bitumen contained therein. Vokac (*16*, *17*) [1] made a comprehensive study of the correlation of physical tests with service behavior and was able to evaluate pavement condition by certain characteristics of compressed specimens. He also found that characteristics of the asphalts recovered from the mixtures could be correlated with pavement condition. Hillman (*3*) has shown that the Hubbard–Field stability of sand-asphalt mixtures is dependent upon the consistency of the contained bitumen at the test temperature and that the relationship varies for different asphalts. His work was done only on unweathered mixtures. Raschig and Doyle (*11*) in studying the flexural strength of asphaltic pavements concluded that the penetration and ductility of the asphalt in the mixture had the greatest influence on the modulus of elasticity and modulus of rupture.

[1] Italic numbers in parentheses refer to the bibliography, p. 94.

Table 1.—*Results of extraction tests and analysis of bitumens recovered before and after exposure in weatherometer*

Mixture designation	Type of molded specimen	Time of exposure in weatherometer	Bitumen extracted	Loss in weight		Tests on recovered bitumen										
				Mixture basis	Bitumen basis	Penetration, 100 gm., 5 sec.				Softening point	Ductility, 5 cm. per min.				Organic insoluble in 86° B. naphtha	Ash by ignition
						at 41° F.	at 59° F.	at 77° F.	at 95° F.		at 41° F.	at 50° F.	at 60° F.	at 77° F.		
COMPOSITE MATERIAL																
		Hours	*Percent*	*Percent*	*Percent*					*°F.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Percent*	*Percent*
A	Hubbard–Field	None	6.6	------	------	18	51	156	----	104.9	84	250+	250+	-	24.14	0.43
A-1	do	100	6.4	0.18	2.7	12	34	94	----	117.0	11	50	120	-	24.30	.41
A-2	do	300	6.2	.29	4.3	----	22	59	148	125.9	5.5	14.5	42	-	27.17	.82
A-3	do	900	6.0	.42	6.3	----	12	29	74	141.8	----	----	8.5	69	30.27	.69
B	Compression	None	6.6	------	------	18	51	156	----	104.9	84	250+	250+	-	24.14	43
B-1	do	100	6.4	.12	1.8	14	41	117	----	112.2	20	200	210	-	24.92	.90
B-2	do	300	6.3	.18	3.2	----	24	65	159	124.4	6.5	19	113	-	26.87	.61
B-3	do	900	5.9	.33	4.9	----	14	34	85	139.3	----	----	12.5	90	30.26	44
MATERIAL PASSING No. 4 SIEVE																
C	Hubbard–Field	None	6.7	------	------	18	50	158	----	105.8	91	168	175	-	24.00	.29
C-1	do	100	6.1	.20	3.1	12	33	92	----	117.3	12	78	165	-	24.81	.66
C-2	do	300	6.1	.29	4.4	----	21	57	142	127.1	6	14.5	115	-	27.90	.57
C-3	do	900	5.9	.42	6.6	----	11	26	64	144.8	----	----	5.5	69	31.62	.79
D	Compression	None	6.7	------	------	18	50	158	----	105.8	91	168	175	-	24.00	.29
D-1	do	100	6.3	.13	2.4	15	42	117	----	113.3	27	230	250+	-	24.63	.54
D-2	do	300	6.1	.22	3.4	----	24	67	168	123.5	6.5	30	128	-	26.49	.65
D-3	do	900	6.0	.33	5.0	----	14	33	81	139.8	----	----	10.5	98	30.71	.63

Table 2.—*Results of stability and compressive load tests on composite mixture before and after weathering, and penetrations of the contained asphalts at test temperatures*

Weathering of specimens tested and test temperature	Hubbard-Field stability		Compressive load	
	Load [1]	Penetration of contained asphalt at test temperature	Load [1]	Penetration of contained asphalt at test temperature
Unweathered, tested at:	*Lbs.*		*Lbs.*	
−10° F.	17,400 (3)	0.8	2,275 (2)	0.8
10° F.	13,150 (3)	2.7	----------	----------
25° F.	----------	----------	1,255 (2)	6.8
40° F.	7,540 (4)	16	700 (2)	16
50° F.	----------	----------	480 (2)	30
77° F.	2,230 (4)	156	165 (2)	156
140° F.	350 (4)	7,000	26 (2)	7,000
Weathered 100 hours, tested at:				
25° F.	11,800 (3)	4.8	----------	----------
40° F.	8,400 (3)	11.5	1,040 (2)	13
77° F.	2,980 (3)	93	335 (2)	117
140° F.	560 (3)	3,300	42 (2)	4,900
Weathered 300 hours, tested at:				
25° F.	12,800 (3)	3.8	----------	----------
40° F.	9,930 (3)	7.4	1,425 (2)	9.3
77° F.	3,820 (3)	59	485 (2)	65
140° F.	715 (3)	1,580	68 (2)	1,630
Weathered 900 hours, tested at:				
−15° F.	21,650 (2)	.4	----------	----------
20° F.	14,850 (2)	1.7	2,640 (1)	1.9
50° F.	9,550 (3)	7.6	1,565 (2)	8.7
77° F.	5,030 (3)	29	845 (2)	34
140° F.	1,155 (2)	680	125 (1)	840

[1] Figures in parentheses indicate the number of specimens for which average values are given.

These and other investigations indicate that there is some correlation between the physical properties of asphaltic mixtures and some of the characteristics of the contained binder. Therefore, this investigation was initiated to determine what correlation existed between the physical properties of bituminous mixtures and the characteristics of the binder before and after weathering, all testing being done under closely controlled laboratory conditions.

CONCLUSIONS DRAWN FROM STUDY

While this investigation is not conclusive, due to the lack of data on the more standard types of asphaltic mixtures, the following trends were observed and appear to be worthy of consideration for future study:

1. The physical properties of a given mixture are dependent upon the consistency of the contained asphalt at the test temperature.

2. The Hubbard–Field stability of a given mixture after weathering is the same as the stability of an unweathered mixture when the test is made at a temperature at which the contained asphalt has the same consistency.

3. Test data from other investigations indicate that there is some characteristic of asphalts which causes comparable mixtures prepared with different asphalts to vary in physical properties even when the contained asphalts are of the same consistency.

4. Since the data from several investigations show that the bitumens extracted from pavements of the type that tend to crack usually have low penetrations and ductilities at 77° F., it would seem logical to make physical tests on asphaltic mixtures at low temperatures in the range corresponding to these penetrations and ductilities and to correlate the results with the properties of the contained asphalts at these temperatures.

5. Since durability of the asphalt in bituminous mixtures is dependent upon the changes occurring in the asphalt during mixing and subsequent weathering, those asphalts that have the greatest resistance to change should be the most durable. This study indicates that changes in the asphalt contained in a mixture can be measured directly by tests on the recovered bitumen or indirectly by changes in the physical properties of the mixture.

6. The durability of asphaltic pavements, as influenced by the consistency of the asphalt, depends upon the climatic environment in which they are located. The critical penetration of the bitumen at which pavement cracking occurs is higher in cold climates than in warm climates.

7. Correlations of the changes occurring in the thin-film oven test with changes in the physical properties of asphaltic mixtures and in the characteristics of the contained bitumen might aid in the development of specification limits based on the thin-film oven test which would insure durable asphalts.

KENTUCKY ROCK ASPHALT IDEAL FOR LABORATORY INVESTIGATION

This investigation was of a preliminary nature and was of limited scope since the test data were obtained on one asphaltic mixture only. Kentucky rock asphalt was selected because of its uniformity, its compactibility at normal temperatures, and the reputed high quality of its contained asphalt. The high penetration of the asphaltic binder and the high percentage of voids in the compressed specimens made this material especially suitable for study under accelerated weathering conditions. The ready compactibility of the rock asphalt without heating provided for uniformity in the characteristics of the binder in the individual test specimens, and the changes incurred during weathering were not masked by changes resulting from the mixing and molding operations that often produce erratic results in weathering tests of hot bituminous mixtures.

Hubbard–Field stability and direct compression were selected as physical tests for this study. Compressed specimens were tested before and after weathering by exposure to ultraviolet light and heat under controlled conditions in a laboratory weatherometer. Two series of tests were made on the rock asphalt—one on the composite material as received, and the other on the portion of the mixture passing the No. 4 sieve. The gradation of the rock asphalt as received was as follows:

	Percent
Retained on ⅜-inch sieve	1.4
Passing ⅜-inch sieve	98.6
Passing No. 4 sieve	90.8

All specimens were compressed at room temperature under a load of 3,000 pounds per square inch, the load being maintained for 2 minutes. Sufficient mixture was used to mold specimens 2 inches in diameter by 1 inch in height for the Hubbard–Field stability test, and 2 inches in diameter by 2 inches in height for the compression test. Volume and weight determinations were made for each compressed specimen and the density calculated as a check on the uniformity of compression. Four sets of stability and compression specimens were made for each of the two mixtures. One set was tested immediately and the other three were exposed to ultraviolet light and heat in a weatherometer. During weathering the temperature was maintained at 140° F. and the relative humidity at 40 to 50 percent. The exposure periods were 100, 300, and 900 hours. The specimens were weighed periodically and loss in weight calculated.

Hubbard–Field stability and compressive strength tests were made initially at 40° F., 77° F., and 140° F. on the unweathered mixtures. As the work progressed additional specimens were molded and tested at other temperatures. The specimens weathered for 100 and 300 hours were tested at 40° F., 77° F., and 140° F. and those weathered for 900 hours were tested at 50° F., 77° F., and 140° F. Tests of the weathered specimens were also made at lower temperatures to provide a more complete picture of the changes in strength that may occur under normal climatic conditions.

After testing for stability and compressive strength the bitumen was extracted with benzol and recovered by the Abson method. The stability and compression specimens were extracted and recovered separately for each mixture and exposure period. The recovered bitumens were tested for penetration and ductility at several temperatures, softening point, organic matter insoluble in 86° B. naphtha, and ash by ignition. The results of the extraction tests, loss in weight during exposure, and tests on the bitumens recovered from the mixtures are given in table 1.

Table 2 gives the average results for Hubbard–Field stability and compressive load for the composite mixture. As seen in table 1, the test results on the material passing the No. 4 sieve were nearly the same as those on the composite material. The stability and compressive load values, which are not reported, were also similar. Therefore, in the

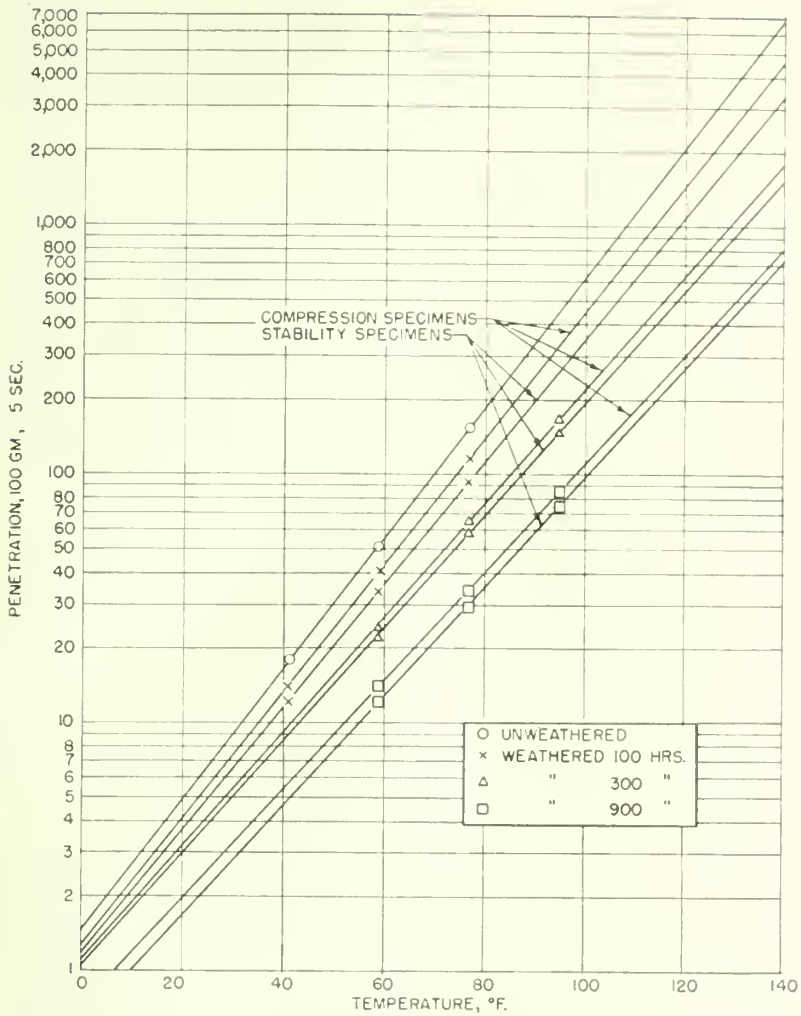


Figure 1.—Effect of test temperature on penetration of bitumens recovered from composite mixture before and after weathering.

following discussion only the test results for the composite material will be considered.

A previous study (*6*) has shown that a linear relationship exists when the log-penetration of an asphalt is plotted against the test temperature and that, by extending this line, it is possible to estimate the penetration of the asphalt at both higher and lower temperatures. In figure 1, the log-penetration–temperature relationships are shown for the bitumens recovered from the composite mixture before and after each period of weathering. From these curves it was possible to estimate the penetration of the bitumen contained in the stability and compression specimens at the temperature at which they were tested. These values also are given in table 2.

In studying the physical properties of bituminous mixtures there has always been the problem of selecting the most suitable temperature to use in making the tests. Endersby (*2*), in discussing the triaxial test, emphasized the need for selecting the most representative test temperature and suggested adjusting this temperature to the climate in which the mixture is used. In this study, temperatures for the stability and compressive load tests were selected so as to cover the temperature range to which the mixtures might be subjected under normal service conditions.

HUBBARD–FIELD STABILITY VARIED WITH TEST TEMPERATURE

The effect of test temperature on Hubbard–Field stability before and after each period of weathering is shown in figure 2. When the log-stability was plotted against temperature a straight-line relation was found to exist for stability values below approximately 10,000 pounds. Further reductions in test temperature caused the stabilities to increase at a lower rate.

Figure 3 shows the effect of test temperature on compressive load, the data being plotted in the same manner as for the stability tests. These curves are straight lines for compressive load up to approximately 1,000 pounds where further reductions in test temperature give lower rates of increase. The test values for compressive load at 140° F. on the unweathered mixture do not fall on the straight-line portion of the curve, but this is probably due to the failure of the compression machine to record such low loads accurately.

The above data show the wide range in the strength of the bituminous mixture which results from changes in temperature and alterations due to exposure and weathering. Yet, pavements constructed of the same type of mixture are carrying extremely heavy traffic without showing displacement in summer or brittleness in winter.

The effect of temperature on the physical properties of a given bituminous mixture can be attributed entirely to the binder. As shown previously, the consistency of the asphalt contained in the stability and compression specimens was determined for each test temperature. To show the effect of the consistency of the binder on the physical properties of the unweathered and the weathered mixture, the

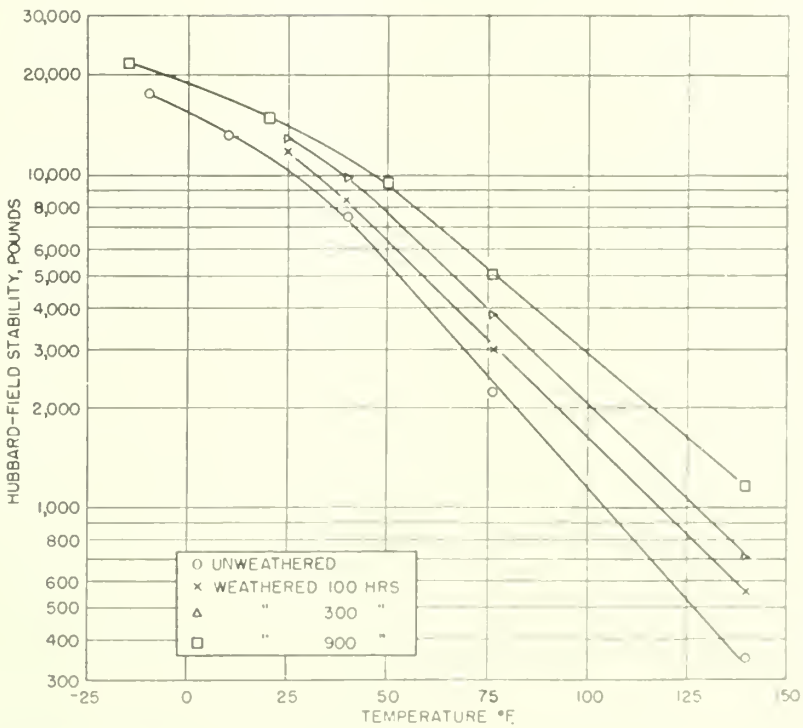


Figure 2.—Effect of test temperature on Hubbard-Field stability of mixture before and after weathering.

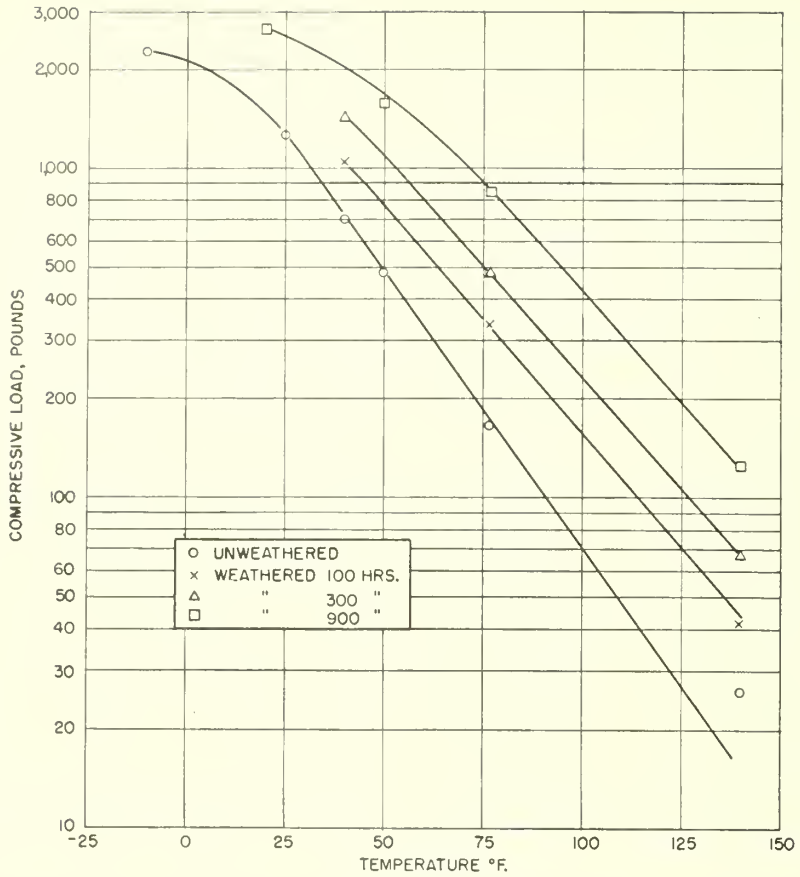


Figure 3.—Effect of test temperature on compressive load of mixture before and after weathering.

log-penetration of the contained asphalt was plotted against the log-stability or log-compressive load at corresponding test temperatures.

Figure 4 shows the stability of the mixture before and after weathering plotted against the penetration of the contained bitumen at the corresponding test temperature. It is of interest to note that all the points, regardless of the amount of weathering and test temperature, fall very close to one curve. A straight-line relation exists for the lower portion of this curve, showing that the rate of increase in stability is proportional to the rate of decrease in penetration. This relation holds until a penetration of approximately 10 is reached where further decrease in penetration causes a lower rate of increase in stability.

This relation between stability and penetration of the contained asphalt not only substantiates the work done by Hillman (*3*) but also shows that the stability of an asphaltic mixture varies with the consistency of the contained bitumen which may be altered by hardening or by reducing the temperature at which the stability test is made.

COMPRESSIVE LOAD VALUES DIFFER FOR SAME CONSISTENCY OF CONTAINED BITUMEN

In figure 5 the compressive load of the rock asphalt before and after weathering is plotted against the penetration of the contained asphalt at the corresponding test temperature. In this figure two separate curves were developed—one for tests on the unweathered mix-

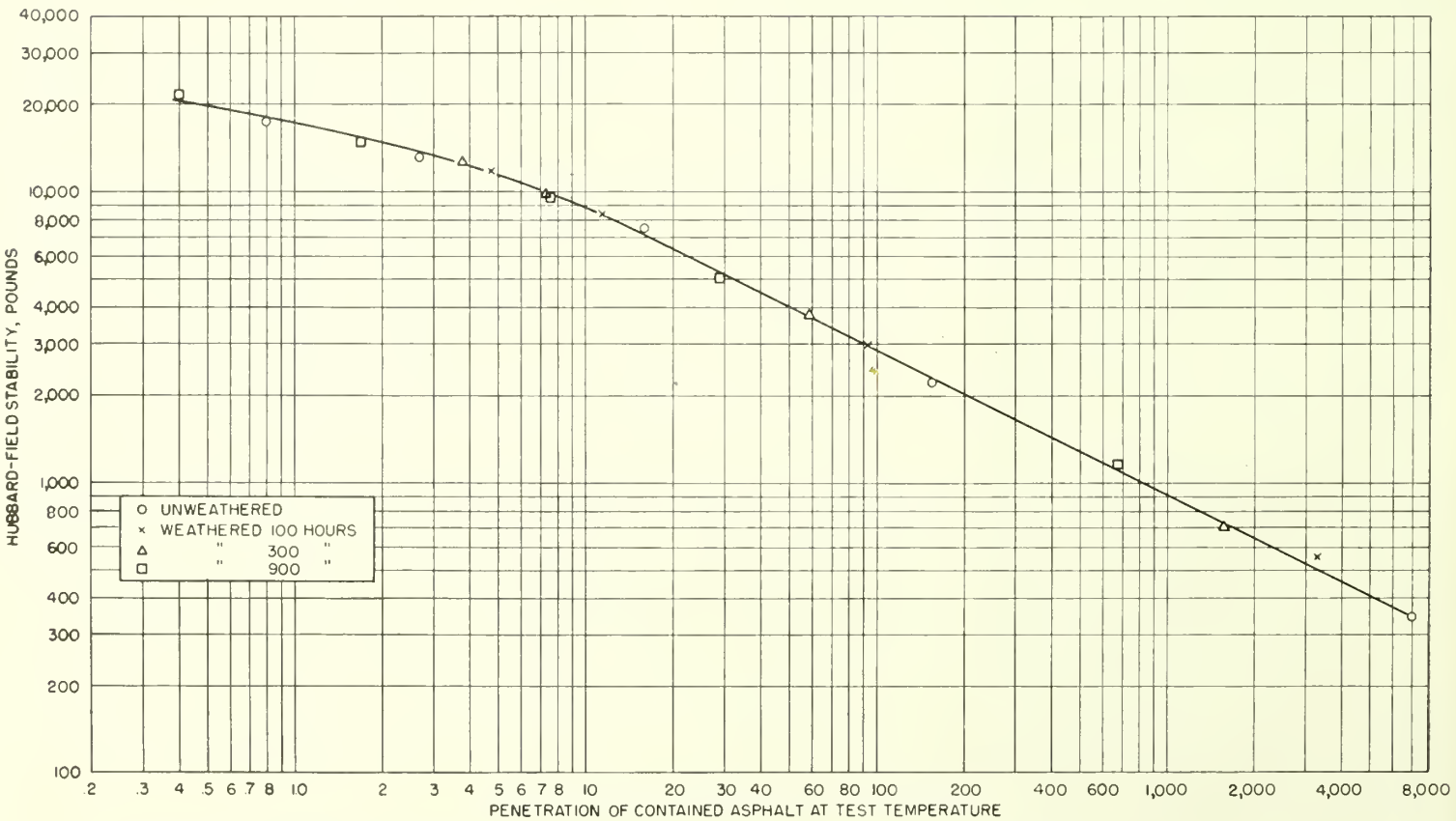


Figure 4.—Relation between Hubbard-Field stability and penetration of contained asphalt at test temperature.

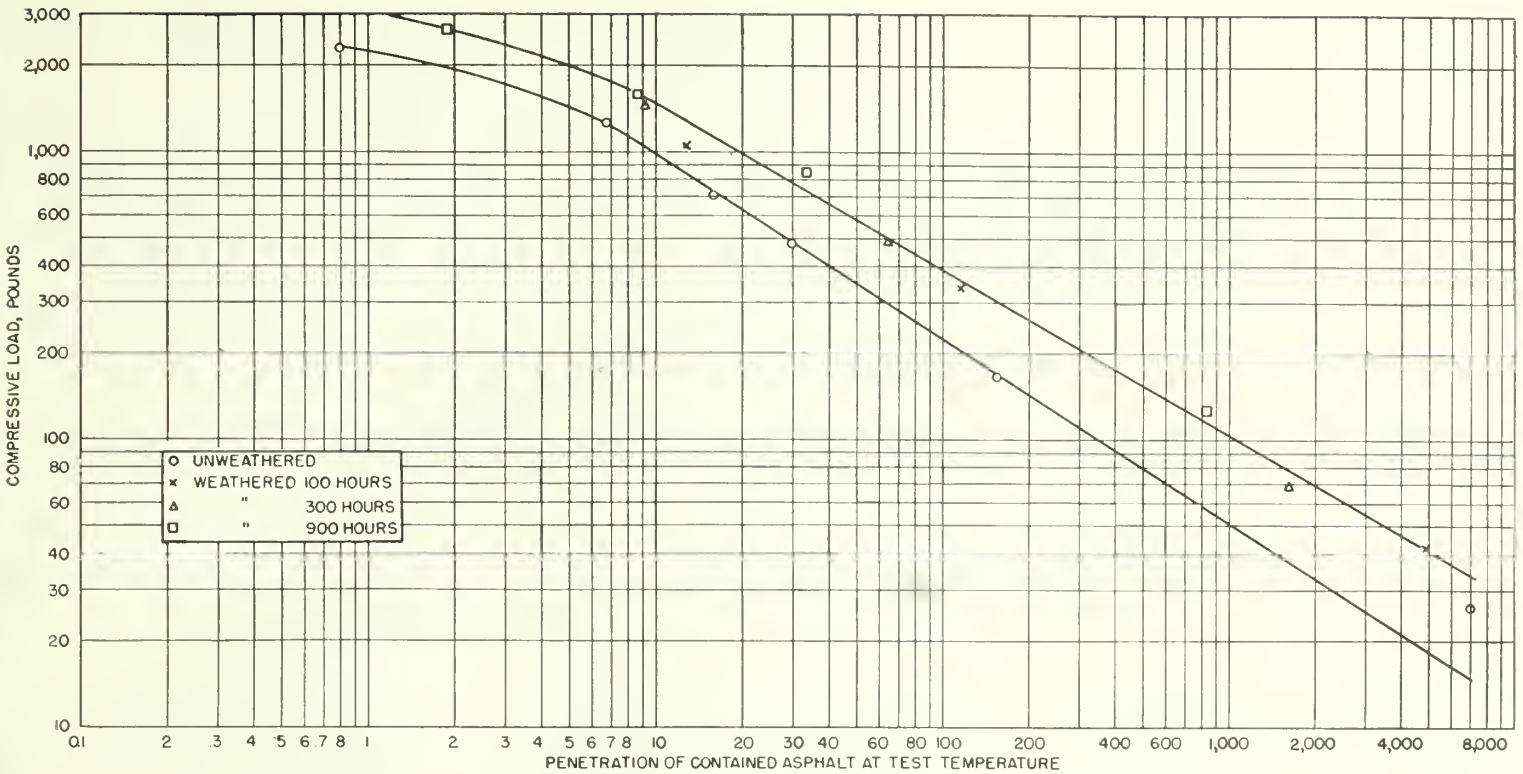


Figure 5.—Relation between compressive load and penetration of contained asphalt at test temperature.

ture tested at different temperatures, and the other for tests on all the weathered specimens tested after the three exposure periods at the different temperatures. The shape of these curves is similar to that in figure 4 for the stability specimens. The lower parts of the curves are straight lines, showing a uniform rate of change in compressive load with change in penetration. Below approximately 10 penetration the rate of increase in compressive load decreases.

The cause of the formation of two separate curves for the compression data while only one was formed for the stability data is not apparent at the present time. It is possible that during accelerated weathering of the molded specimens an outer crust was formed, and when tested in compression the circumferential hardening gave additional support. Due to the manner in which the stability specimens are tested, this outer crusting probably would not affect the values to any extent. The relatively high loss of volatile material from the asphalt in the rock asphalt would accelerate the outer hardening. Similar work on mixtures prepared with standard asphalt cements may aid in showing the cause for the formation of separate curves for the unweathered and weathered mixture when tested in compression.

Vokac (*15, 16*) has shown that in the compression testing of bituminous mixtures the fundamental characteristics of modulus of elasticity and elastic limit may be readily obtained and applied to the study of mixtures under service conditions. In making the compression tests on the rock-asphalt mixtures used in this investigation, load-deformation curves were obtained on the compression specimens before and after weathering for 900 hours, and the modulus of elasticity calculated for each condition. The modulus of elasticity is the unit stress divided by the unit strain within the elastic limit or over that portion of the stress-strain curve that is a straight line. The curves formed for the rock-asphalt specimens had some curvature at the beginning of

Table 3.—*Modulus of elasticity of compression specimens before and after weathering.*

Weathering of specimens tested and test temperature	Modulus of elasticity	Penetration of contained asphalt
	Lbs per sq. in.	
Unweathered, tested at:		
−10° F.	49,250	0.8
20° F.	14,850	5.0
50° F.	5,500	30
77° F.	2,200	156
Weathered 900 hours, tested at:		
20° F.	46,350	1.9
50° F.	19,650	8.7
77° F.	10,200	34

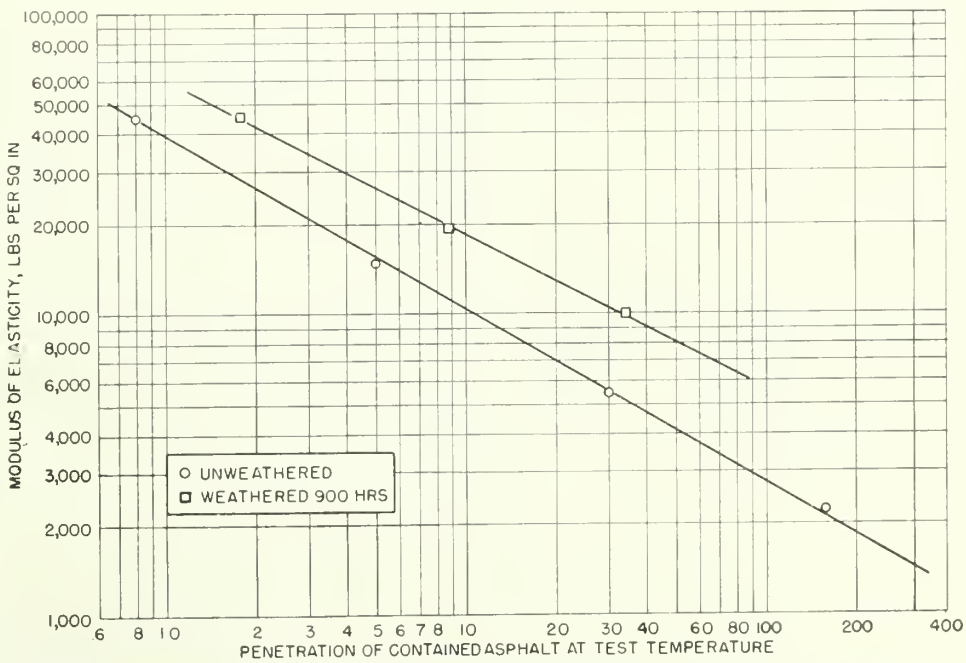

MODULUS OF ELASTICITY, LBS PER SQ IN
PENETRATION OF CONTAINED ASPHALT AT TEST TEMPERATURE
UNWEATHERED
WEATHERED 900 HRS

Figure 6.—Relation between modulus of elasticity and penetration of contained asphalt at test temperature.

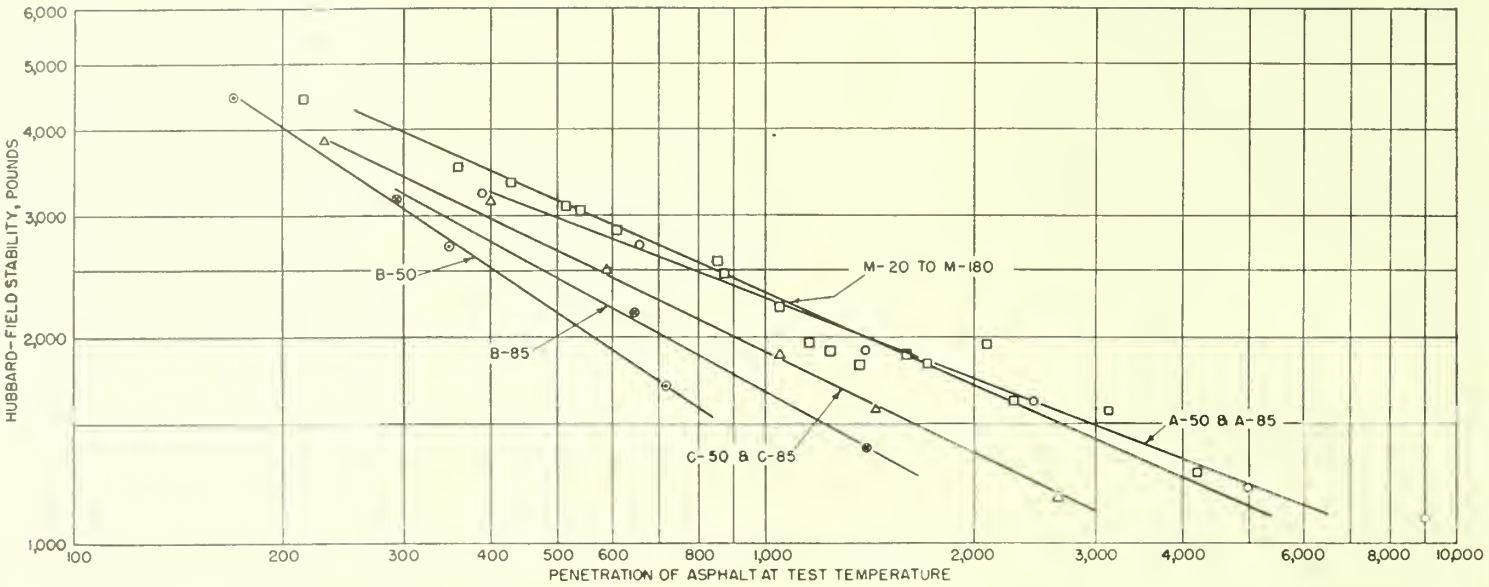


Figure 7.—Relation between Hubbard-Field stability and penetration of contained asphalt for asphalt from various sources.

the loading and then approached a straight line until the elastic limit was reached. Since these curves did not have a definite straight portion, and since some of the loads and deformations were of low magnitude, the moduli were calculated by taking the middle 75 percent of the total compression load and the corresponding deformation from the curves.

The modulus of elasticity values at the various test temperatures for the unweathered mixture and the mixture weathered for 900 hours are given in table 3, together with the penetration of the contained asphalt at the test temperature.

By plotting the modulus of elasticity against the penetration of the contained asphalt at the test temperature, both on log scale, relations were found as shown in figure 6. Two straight-line curves were formed, one for the unweathered mixture and the other for specimens weathered for 900 hours. This relation is similar to that for the penetration-compressive load curves shown in figure 5, except that the straight lines do not break when the penetration of the contained asphalt is reduced to approximately 10.

The effect of consistency of the binder on the physical properties of mixtures has been included in test data from other investigations, although in some cases no attempt was made to correlate the two, and a further study of these data is of interest.

In Hillman's study (*3*) of the effect of consistency on stability, data were presented for stabilities and penetrations of the contained bitumen at temperatures of 104° F., 122° F., and 140° F. for asphalts from several sources. The relations between stability and penetration of the contained bitumen at the test temperature for some of these asphalts are shown in figure 7. Although the tests were made only at the high temperatures and cover a relatively narrow range, there was considerable difference between the stability-penetration relationships for the various asphalts.

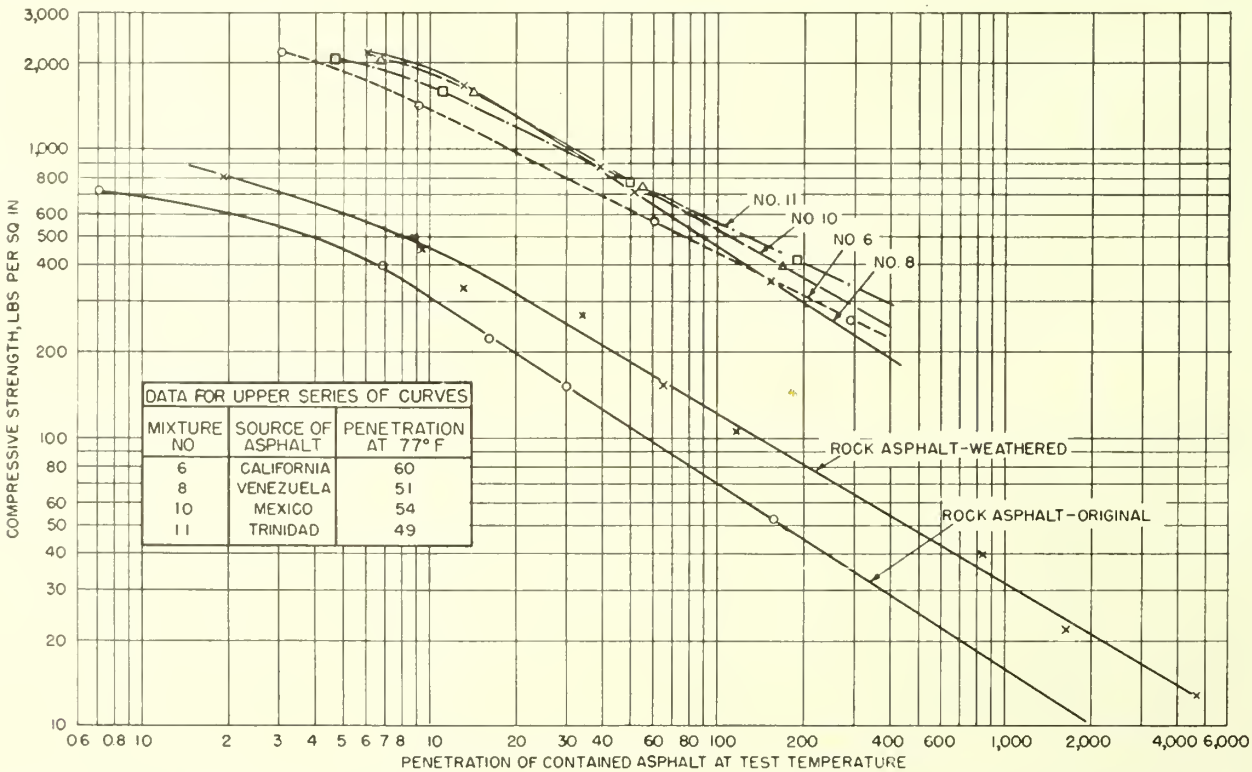

COMPRESSIVE STRENGTH, LBS PER SQ IN
PENETRATION OF CONTAINED ASPHALT AT TEST TEMPERATURE
NO. 11
NO 10
NO 6
NO. 8
ROCK ASPHALT-WEATHERED
ROCK ASPHALT-ORIGINAL
DATA FOR UPPER SERIES OF CURVES
MIXTURE NO | SOURCE OF ASPHALT | PENETRATION AT 77° F
6 | CALIFORNIA | 60
8 | VENEZUELA | 51
10 | MEXICO | 54
11 | TRINIDAD | 49

Figure 8.—Relation between compressive strength and penetration of contained asphalt at test temperature for asphalts from various sources.

DATA OF OTHER INVESTIGATIONS CONSIDERED

Vokac (*17*), in studying the thermal susceptibility of asphaltic paving mixtures, made compressive strength tests at 34° F., 77° F., and 140° F. on mixtures containing asphalts from several sources. Although only the effect of temperature on compressive strength was given, penetration data were taken from another investigation (*6*) for similar asphalts and correlated with the compressive strength values at the three temperatures. The penetration-compressive strength relations for some of these asphalts are shown in figure 8. Although the compression tests were made on mixtures of the sheet-asphalt type, the shape of the curves is similar to those for the rock asphalt used in this investigation. Asphalts from different sources showed somewhat different relationships.

Figure 9 shows some penetration-compression data taken from a report by the Barber Asphalt Corporation (*1*). Here compressive strength tests were made on a sheet-asphalt mixture at three temperatures, using asphalt cements from several sources. Penetrations were reported at three temperatures so that the consistency of the contained asphalt at the temperatures of the compression test could be determined. As in figure 8, the straight-line portion of the curves for the various asphalts shows some spread.

The data illustrated in figures 7, 8, and 9 show that there is some characteristic of an asphalt other than consistency that causes mixtures made with different asphalts to vary in stability and compressive strength.

In studying the effect of temperature on shear characteristics of bituminous paving compositions Skidmore (*13*) noted that as the test temperature was reduced, the rate of increase in shear strength decreased and for some asphalts the shear strength decreased rather than increased. Some of Skidmore's data are plotted in figure 10 to show the effect of temperature on the shear strength. From this study Skidmore concluded that:

> "1. The inherent characteristics and quantity of the bitumen in the mixture are much more important at low temperatures than at normal and higher temperatures in the pavement.
>
> "2. There is an optimum proportioning of the several ingredients of the paving mixture that will insure the best performance over the entire range of temperatures.
>
> "3. In regions where low temperatures obtain, the adoption of considerable softer bitumens than have been used commonly, will constitute one of the most important advances in the direction of better pavements."

Oliensis (*8*), in discussing mechanical tests on mixtures, pointed out that due to the difference in susceptibility of asphalts either relatively good or relatively poor test results can be obtained for mixtures with any given asphalt, provided the particular temperature selected for the designated mechanical test has a favorable or unfavorable influence on the strength of the mixture because of the consistency of the binder at the test temperature. He also noted that all laboratory tests proposed thus far for determining the quality and durability of mixtures or the contained asphalt are in essence tests of consistency or tests in which consistency plays a predominant role.

He further stated:

> "* * * Hence, if reliable durability tests are to be developed in the laboratory on the finished mixture, our research engineers must bend their efforts to devising other tests, in which either the varying consistency of the incorporated asphalt plays no role whatever, or in which the role it does play is frankly recognized and allowed for by having both the compaction and testing processes conducted at such temperatures (perhaps different ones for mixtures containing different asphalts) as will insure equal compaction, and equal consistency of the bituminous binders at the moment when the mixtures are being tested."

Vokac(*16*, *17*), in analyzing the data from the study of the correlation of physical tests on asphalt mixtures with service behavior, showed that, in general, those mixtures with the highest compressive strengths, Hubbard–Field stabilities, and moduli of elasticity were usually from pavements of the type that tend to crack. The asphalt extracted from these pavements, in general, had the lowest penetration and ductilities and the highest softening points. The mixtures representing the sound pavements had lower compressive strengths, stabilities, and moduli of elasticity with corresponding higher penetrations and ductili-

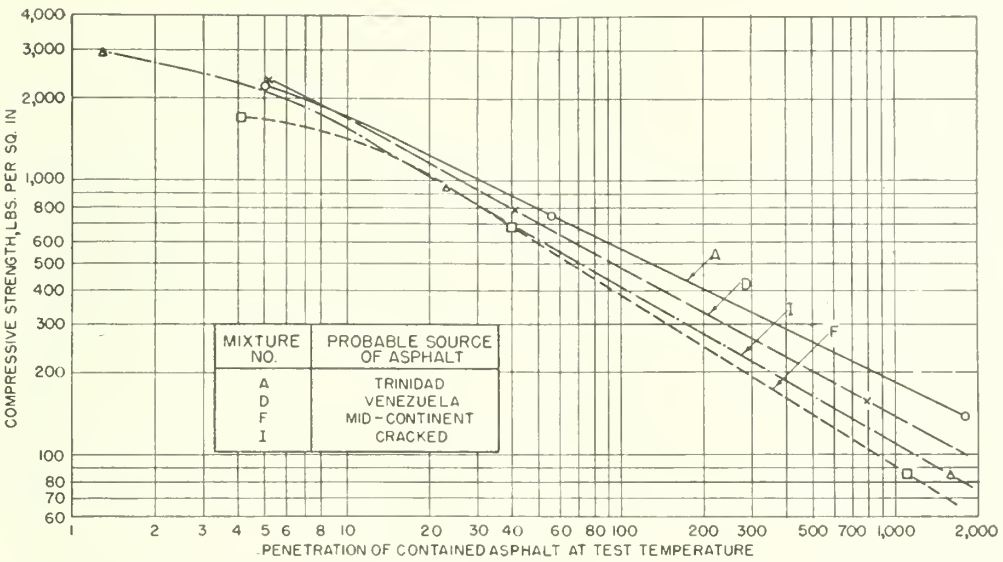


Figure 9.—Relation between compressive strength and penetration of contained asphalt for asphalts from various sources.

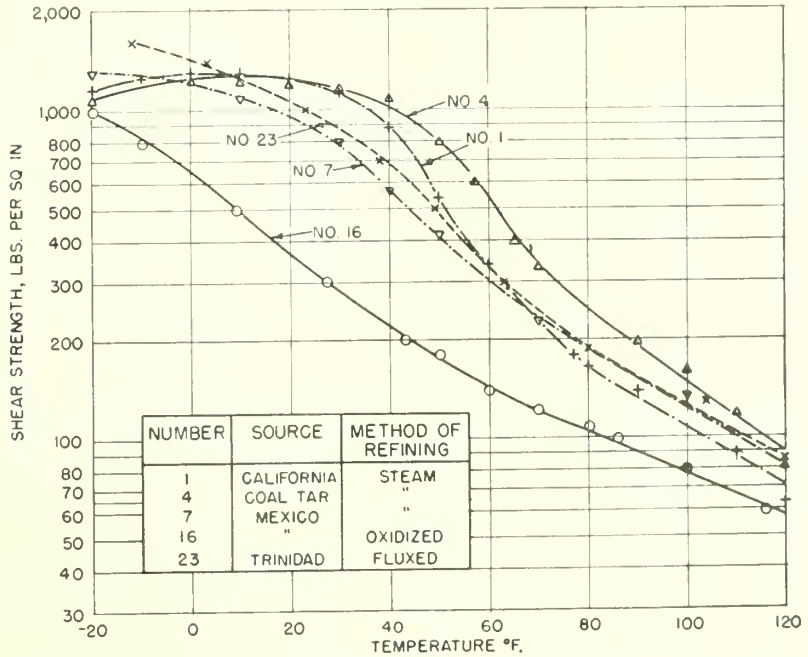


Figure 10.—Effect of test temperature on shear strength of mixtures containing various bituminous materials.

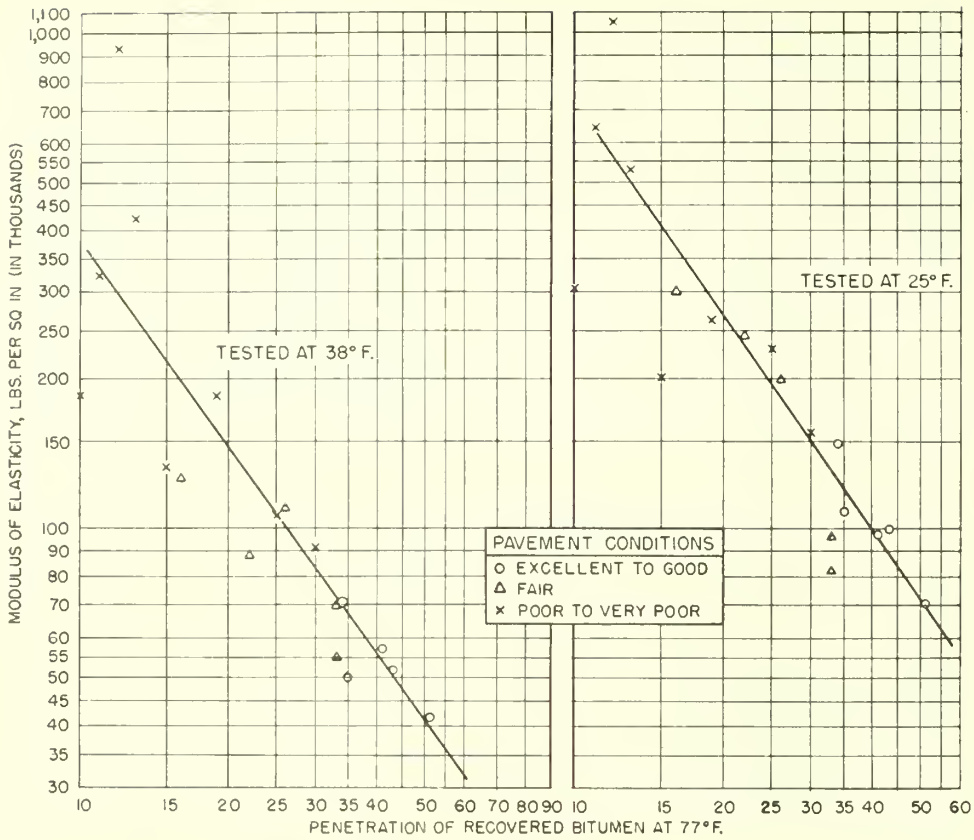


Figure 11.—Relation between modulus of elasticity and penetration of contained asphalt for pavement samples.

ties and lower softening points of the contained asphalt.

In an investigation of the behavior of asphalts in paving mixtures, Raschig and Doyle (*11*) concluded that the penetration and ductility of the asphalt in the mixture had the greatest influence on the modulus of elasticity and modulus of rupture and, in general, the condition of the pavement was poorest where the penetration and ductility were lowest. The physical properties of the mixtures were determined by flexural tests on beams cut from pavement samples. Although the tests on the beams were made at 25° F. and 38° F. and the penetration of the recovered asphalt was made at 77° F., a good correlation of these properties was obtained. Figure 11 shows the relation between the moduli of elasticity at 25° F. and 38° F. and the penetration of the contained asphalt at 77° F.

CONSISTENCY VARIES WITH CLIMATIC CONDITIONS

Shattuck (*12*), Hubbard and Gollomb (*4*), Powers (*9*), Thomas (*14*), and others also have shown that pavement conditions can be correlated with penetration of the contained asphalt, and some of these investigators have shown the correlation of ductility and softening point of the asphalt with durability. A summary of the data presented by these investigations is given in table 4.

It is apparent from these investigations that the characteristics of the asphalt contained in the mixture are of fundamental importance in studying the durability of asphaltic pavements. Since durability is dependent upon the ultimate hardness of the contained asphalt, usually measured at 77° F., the range in temperatures to which pavements are subjected also should be considered.

The pavement samples analyzed by Shattuck, Thomas, Hubbard, and Vokac were taken from the northeastern and north-central part of the United States where severe cracking of the pavements occurred when the asphalt had hardened to a penetration of 20. However, Powers noted, in Arizona, that where the penetration of the contained asphalt was above 10 the pavements subjected to Arizona climate were free from cracking. The analysis of asphaltic concrete mixtures from Cuba by the Public Roads Administration (*10*) showed that the average penetration of asphalts from pavements in good condition was 9 and for cracked pavements the average penetration was 5.

The minimum ductility and maximum softening point of the recovered asphalts, as related to the cracking of pavements, also appear to be dependent upon the climatic temperatures to which pavements are subjected. Although ductility and softening point are inherent characteristics of an asphalt, asphalts from various sources have widely different penetration-ductility and penetration-softening point relationships.

In the studies cited above, the characteristics of the recovered asphalts were those at the time of pavement rating. Very few data are available that give the inherent characteristics of the original asphalts and particularly the changes in the asphalts due to plant mixing and laying. Because the characteristics of the asphalts contained in the mixture at the beginning of the pavement life are not known, changes in the inherent characteristics due to the age of pavement cannot be ascertained. Consequently, the relative resistance of the various asphalt cements cannot be evaluated.

For the natural rock-asphalt mixture studied in this investigation, the correlation of the physical properties of the mixture with the penetration of the contained asphalt shows that for a given mixture the physical properties are dependent on the consistency of the asphaltic binder at the test temperature. This finding is substantiated by similar correlations of the data from other investigations. However, it is well known that when different asphalts are exposed to the high temperatures encountered in plant mixing, weathering un-

Table 4.—*Correlation of pavement condition with physical properties of recovered bitumen*

Source of data	Location of pavements	Condition of pavement	Tests on recovered bitumen		
			Penetration 77° F.	Ductility 77° F.	Softening point
				Cm.	° *F.*
Shattuck (*12*)[1]	Detroit, Mich.	Very good	20+	50+	----
		Good	20+	25+	----
		Badly cracked	20−	25−	----
Thomas (*14*)	Minnesota	Good [2]	41	----	----
		Fair [2]	26	----	----
		Poor [2]	20	----	----
Hubbard and Gollomb (*4*).	Ohio, Mich., N. Y., Ind., D. C.	Sound	30+	----	----
		Prone to crack	30−	----	----
		Cracking type	20−	----	----
Vokac (*16, 17*)	Ohio, Pa., Md., Va., Mo., Ill., Ind., Mich., N. Y., N. J., D. C.	Sound	25+	24+	----
		Prone to crack	18–25	4–24	160−
		Cracking type	18−	4−	160+
Powers (*9*)	Arizona	Good	10+	10+	160−
		Cracked	10−	10−	160+
Public Roads Administration (*10*)	Cuba	Good [2]	9	1.5	199
		Cracked [2]	5	0.5	217

[1] Italic numbers in parentheses refer to the bibliography, p. 94.
[2] Average values.

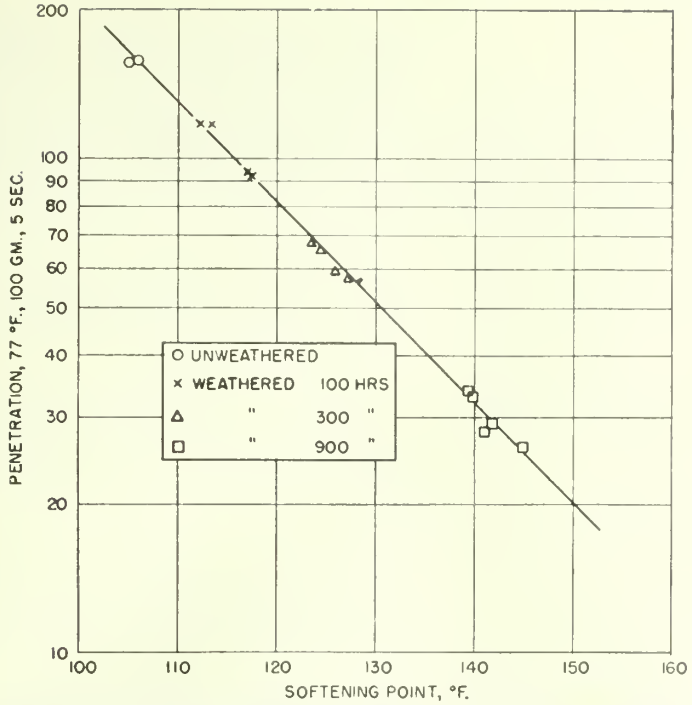


Figure 12.—Relation between softening point and penetration of bitumen recovered from mixtures before and after weathering.

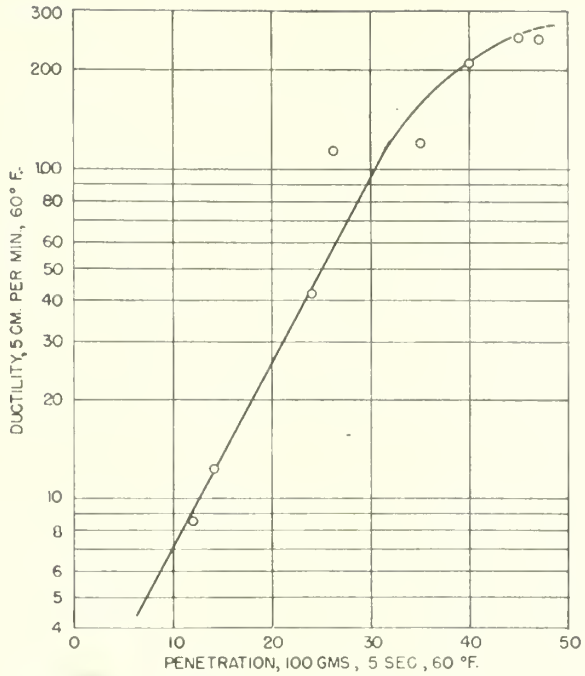


Figure 13.—Relation between penetration and ductility of bitumen recovered from mixtures before and after weathering.

der service conditions, or exposure to any of the various laboratory oxidation tests, the rate of change in their characteristics is a variable. Therefore, it is to be expected that the relation between the physical properties of asphaltic mixtures and the properties of the asphaltic binder will vary with different asphalts and that it will also be affected by the type and grading of the aggregate, the degree of heat used in mixing, degradation of the aggregate during consolidation, and the degree of consolidation. It seems probable that in some of the studies that have been made of the durability of bituminous mixtures, variables such as these may have masked the changes that have occurred in the asphalt binder.

RETENTION OF ORIGINAL CHARACTERISTICS A MEASURE OF DURABILITY

Previous investigations (*5*, *7*) have shown that certain correlations between penetration, ductility, and softening point are useful in studying changes induced in asphalts by weathering under actual service conditions or by oxidation tests in the laboratory. When an asphalt is subjected to oxidation the decrease in penetration is accompanied by an increase in softening point and usually by a decrease in ductility. After various periods of weathering or oxidation it has been shown that when the softening points of the various residues are plotted against the corresponding log-penetrations smooth curves are formed which, for many asphalts, are straight lines. If the penetrations are plotted against the log-ductility of the various residues smooth curves can be drawn through the points. For each asphalt these relations exist either for residues obtained from laboratory oxidation tests or bitumens recovered from mixtures after exposure to heat or actual service conditions.

The penetration, softening point, and ductility results for the bitumens recovered from the unweathered and the various weathered rock-asphalt specimens of this investigation are shown plotted in the above manner in figures 12 and 13. In figure 12, all the points formed by plotting softening point against the log-penetration fall very close to a straight line. Because of the softness and the high ductility of the recovered bitumens, it was necessary to determine ductilities at temperatures lower than 77° F. to show changes in the asphalt induced by weathering.

Figure 13 shows penetration plotted against log-ductility, both at 60° F., for the bitumens recovered from the mixture before and after weathering. As compared to many asphalt cements the ductility of the asphalt recovered from the rock-asphalt mixture is high for a given penetration both before and after weathering.

Previous studies (*5*, *7*) have shown that changes occurring in asphalt cements during mixing and after exposure of the mixture under service conditions can be predicted by heating the asphalt in a ⅛-inch film at 325° F. for variable periods of time. The bitumen recovered from the unweathered rock-asphalt mixture used in this study was subjected to the thin-film oven test for 5 hours and the residue tested for penetration, softening point, and ductility. The characteristics of the residue were very similar to those of the bitumen recovered from the stability specimens exposed for 900 hours in the weatherometer, as may be seen in table 5.

Table 5.—*Comparison of tests on asphalts subjected to thin-film oven test and weatherometer exposure*

Material tested	Penetration 77° F.	Softening point	Ductility 77° F.
		°F.	*Cm.*
Thin-film oven test, 5 hours	28	141	81
Weatherometer exposure, 900 hours	29	142	69

The increase in organic matter insoluble in 86° B. naphtha can also be used to determine changes in asphalts caused by exposure to accelerated laboratory oxidation tests or to weathering under service conditions. In studying the effect of the thin-film oven test on 85–100 penetration asphalts (*7*) from nearly all sources, it was found that the naphtha-insoluble matter in the residues ranged from 107 to 139 percent of the insoluble matter in the original materials. Table 1 of this report shows the naphtha-insoluble matter in the bitumens recovered from the rock-asphalt mixtures before and after weathering. The unweathered asphalt from the composite mixture contained 24.14 percent insoluble matter. After weathering for 900 hours this had increased to 30.27 percent for the asphalt in the stability specimens and 30.26 percent for the asphalt in the compression specimens. These increases expressed in terms of the weight of original material were 117 percent for the bitumen recovered from the stability specimens and 119 percent for the compression specimens.

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The Analysis of Motor-Vehicle Registration Records

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH PUBLIC ROADS ADMINISTRATION

Reported by G. P. ST. CLAIR, Chief of Division and SIDNEY B. CLARK, Statistician

This study of the District of Columbia motor-vehicle registration and title records was made primarily for the purpose of developing sampling techniques that would be applicable in all States. The results indicate that the use of standard sampling procedures, adapted specifically to the registration system in the individual State, will produce accurate data regarding the numbers of motor vehicles in existence or in use at a given time and their distribution by year model, weight, body type, and other characteristics. Analysis of certificate-of-title records provides useful information regarding the motor-vehicle market, including the volume and fluctuation of new- and used-car sales, the interstate migration of motor vehicles, and dealers' inventories of used cars.

INFORMATION regarding the motor vehicle, its life, and its habits is of the utmost importance in highway research. It is of great value also to motor-vehicle manufacturers, suppliers, and users and to those who are interested in observing economic trends. A virtual gold mine of such information is found in the records of the State motor-vehicle departments. Unfortunately, this treasure is of the type for which we must dig if we are to find pay dirt. The motor-vehicle departments maintain and use these records primarily for purposes of administration and law enforcement. Motor-vehicle administrators are becoming increasingly aware of the value of the statistics at their command; and a number have installed the most modern methods of record-keeping. They have also been very cooperative in making their files available for study. The fact remains, however, that the adaptation of motor-vehicle records and statistics to the uses of highway research and planning is the responsibility of the agencies engaged in this work. All that can be asked of the motor-vehicle departments is a willingness to cooperate in the endeavor, within the limits of their statutory powers and duties.

Although published registration statistics are valuable if interpreted properly, they fail to give answers to some of the most important questions regarding the motor vehicle. They do not tell us how many vehicles are in existence at a given time, although they are often improperly used for that purpose. They do not tell us how many have been scrapped during a year or the number which have been moved from one State to another. Nor do they give us accurate information from which the age-distribution and service-life characteristics of motor vehicles can be determined. All of this information, however, is contained in the registration and title records, or can be derived, at least approximately, from an analysis thereof. Because of the very large number of motor vehicles, particularly passenger cars, registered in most States, there is an opportunity for the use of sampling methods to obtain at reasonable expense accurate estimates of the items of information most desired.

In order to develop techniques for such a sampling procedure, arrangements were made with the District of Columbia Department of Vehicles and Traffic for the conduct of a pilot study of its registration and title records. This study was in three parts: First, a sample study of private passenger-car registrations in the registration year 1943; second, a sample study of all motor-vehicle registrations in the registration year 1944; and third, a sample study of certificates of titles issued during the calendar year 1941. In the District of Columbia the annual registration period begins on March 1, although registrations for the new year are accepted prior to that date. The plates of the previous year are valid through the month of March.

THE REGISTRATION STUDIES

The registration records of the District of Columbia consist principally of three files: An alphabetical or name file; a serial or plate-number file; and a motor-number file. The alphabetical and serial files are maintained on an annual basis. The motor-number file is not chronological in any sense. The cards are filed by engine number and upon them is recorded the ownership history of every vehicle titled in the District of Columbia since the inception of the file.

In the District of Columbia, as in about 20 of the States, the procedure on transfer of ownership of a vehicle is for the original owner to retain the number plates and attach them to his new vehicle, if he acquires one. The result is that the District of Columbia registration record is a record of ownerships,

GLOSSARY

Original registration: The first registration of a vehicle in a given year.

Subsequent registration: Any registration of a vehicle, other than the original registration, during a given year.

Renewal: The registration, in a given year, of a vehicle which was in registration status at the close of the previous year.

- *Regular renewal:* A renewal made by the same owner.
- *Cross-over renewal:* A renewal made by a different owner from the one who registered the vehicle in the previous year.

Withdrawal: A vehicle withheld from registration but which in some subsequent year will be returned to the registration lists.

Return: The registration of a former withdrawal, i. e., that of a vehicle which had not been registered in the next previous year but had been registered in a year prior to that, and had not lost title status in the State.

Immigration: A vehicle moved in from another State and not registered in the new State of residence prior to the year under study.

- *Duplicating immigration:* A migrant vehicle registered in both the State of origin and the State of destination in the same year.
- *Cross-over immigration:* A migrant vehicle registered in the State of origin in one year and in the State of destination in the next year.

Transfer: A transaction by which a number plate is changed from one vehicle to another.

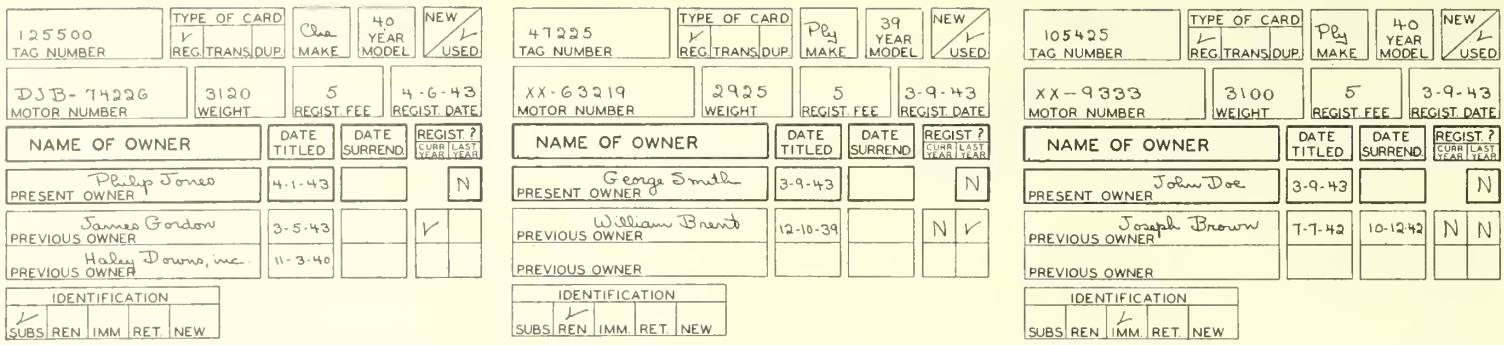

Figure 1.—Enumeration card used in study, illustrating identification of a subsequent registration, a cross-over renewal, and an immigration.

rather than of vehicles; for the same number plate may represent two or more vehicles registered by the same owner in a single year. The transaction represented by the change of a number plate from one vehicle to another is called a transfer. The serial file, therefore, contains one or more cards for each number plate issued. The first of these cards represents the first registration of a vehicle by the given owner. In addition there may be one or more transfers, and there may also be so-called duplicate cards, which represent minor transactions of little consequence.

Since the objective of the study was to obtain information regarding vehicles rather than about their owners, it was necessary to find a means of drawing a representative sample of vehicles from a file of ownerships. To resolve this difficulty, the concept of originals and subsequents was developed. An *original* was defined as the first registration of a vehicle in the given year. A *subsequent* was defined as any registration of a vehicle other than the first. For example if, on March 1, an owner renews the registration of the vehicle he registered in the previous year, that is considered an original registration. If, on July 1, he records the trade-in of the old car for a brand new one, that also is an original registration, although it will appear in the records as a transfer. On the other hand, if he trades in for a used car which had been registered in the same year by the previous owner, that transaction is considered a subsequent registration, although it too would appear on the records as a transfer. If a person not previously owning a car buys and registers a used car which had already been registered by the previous owner within the year, that is considered a subsequent, although it appears in the records as a registration. Thus an original or a subsequent may appear at any point in the sequence of registration and transfer cards under a given serial number.

It may be noted, parenthetically, that, in a State where the plates go with the car on transfer of ownership, the serial or plate-number file may be sampled directly, without recourse to the concept of originals and subsequents. This procedure is, however, an essential step in the case of a State where the plates are retained by the registrant.

Once this concept had been introduced, it became apparent that if a representative sample of all cards in the serial file were drawn, and each card (other than duplicates) was identified as either an original or a subsequent, the cards representing originals would constitute a representative sample of the number of vehicles registered during the year. This identification could not be made on the cards as they were drawn from the serial file. It was necessary to make a search in the motor-number file, the alphabetical file of the previous year, and other available files and records in order to make the complete identification desired.

It was first decided to draw every *n*th card in the serial file, but it soon developed that the job would be much easier if all the cards under every *n*th serial number were drawn. This procedure involved a species of cluster sampling, since a given serial number might contain from 0 to 4 or more cards. The absence of a card under a given serial number meant that no registration was issued under that number during the year. There was a relatively large number of such cases in the years 1943 and 1944, because the District of Columbia was still using 1942 number plates with annual tabs to indicate registration in the given year. Thus, if a vehicle was registered under a given serial number in 1942 and then, because of being scrapped or moved out of the District was not registered in 1943 or 1944, no card would be found under that serial number in the registration file for the later year.

It was of interest, for the sake of economy, to select as small a sample as would give the desired accuracy, which was conceived in terms of a relative error of no more than 2 percent (at the 95 percent level of significance) for the estimates of total vehicles registered. In the 1943 study a 4-percent sample (every 25th serial number) of private passenger-car registrations was drawn, producing a total of 6,144 individuals. In the 1944 study a 5 percent sample, consisting of 8,324 individuals, was taken. For the other vehicle types the relative size of sample varied from 30 percent, or 6,167 individuals, in the case of trucks, to 100 percent in the case of busses and other types of which the total was small.

In addition to the segregation of originals from subsequents, it was desired to obtain certain descriptive information about the vehicles registered during the year; and for this reason the process of identification included the determination of whether each original was a renewal, a return, a new car, or an immigration. A *renewal* is defined as the registration, in a given year, of a vehicle which was in registration status at the close of the previous year. Renewals may be further subdivided into *regular renewals*, i. e., those made by the same owner, and *cross-over renewals*, defined as renewals made by a different owner from the one who registered the vehicle in the previous year. Ordinarily, such transactions occur during the overlapping registration period

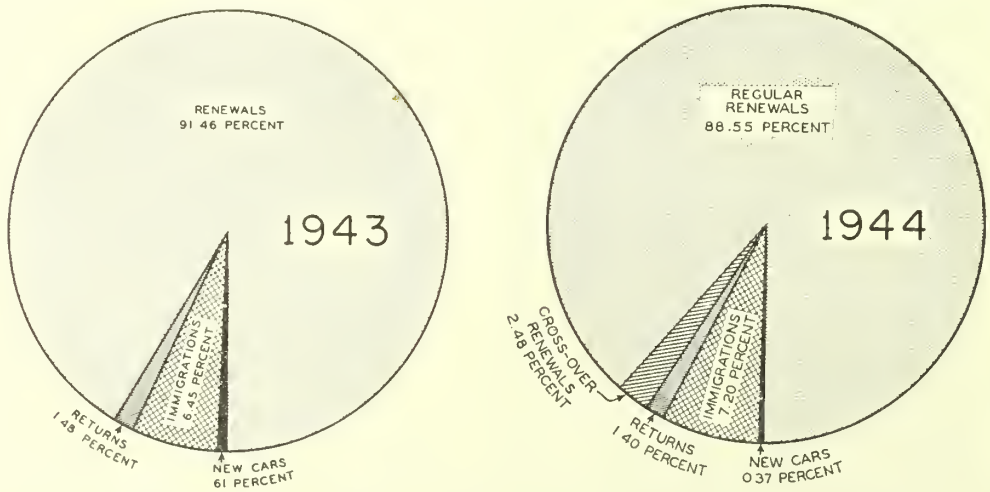

Figure 2.—Relative distributions of private passenger cars registered in 1943 and in 1944.

when new registrations are being issued while plates for the expiring year are still valid.

Each year there is a small number of vehicles that are withheld from registration by their owners, later to be returned to the registration lists: these are called *withdrawals*. A *return* represents the registration of a former withdrawal, i. e., that of a vehicle which was not registered in the next previous year, but which had been registered in a year prior to that, and had not lost title status in the District of Columbia during the interval between registrations. In this study the assumption was made that the returns of one year equal the withdrawals of the preceding year.

An *immigration* is defined as a vehicle moved in from another State and not registered in the District of Columbia prior to the year under study. Immigrations are of two kinds: *duplicating immigrations*, defined as migrant vehicles which were registered in both State of origin and State of destination in the year under study; and *cross-over immigrations*, consisting of migrant vehicles registered in the State of origin in one year and in the State of destination in the next year. If all States made studies of this sort it would be possible, by the exchange of information, to distinguish accurately between duplicating and cross-over immigrations. In the District of Columbia study, cross-over immigrations were taken as equalling all immigrations for which titles were issued prior to March 31, plus one-half of those titled in April. This assumption was influenced by the known registration practices of the adjacent States of Maryland and Virginia.

The identification process is illustrated in figure 1, which shows the type of enumeration card used in the study. Figure 1*A* illustrates the identification of a subsequent. Philip Jones titled the car on April 1, 1943, and registered it on April 6. The symbol N in the box headed "Registered last year?" indicates that he did not register this car in 1942. The next previous owner, James Gordon, titled the car on March 5, 1943, and registered it in the current year, 1943. Since this car was previously registered in the current year by James Gordon, the sample card represents a subsequent registration.

Figure 1*B* illustrates the identification of a cross-over renewal. The present owner did not register the car in 1942, but the next previous owner did. The transfer of title occurred during the cross-over period, in the month of March.

Figure 1*C* illustrates the identification of a cross-over immigration. That it was an immigration is proved by the fact that it was first titled in the District of Columbia as a used car. Because it was first registered in the District in 1943, it is by definition a 1943 immigration. It was, however, first titled by Joseph Brown on July 7, 1942. Therefore, since it was in the District of Columbia at the opening of the 1943 registration year, it qualifies as a cross-over immigration.

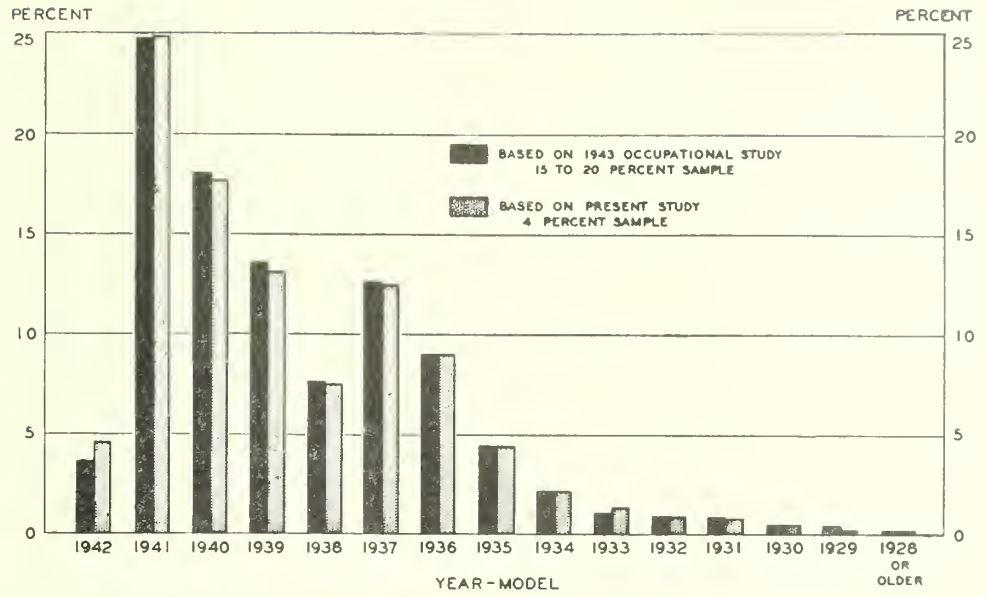

Figure 3.—Year-model distributions of private passenger cars registered in 1943.

A brief summary of the results of the registration studies is given in figures 2–4. The left-hand chart in figure 2 gives the percentage distribution of private passenger cars registered in 1943 with respect to renewals, returns, new cars, and immigrations. More than 91 percent of the passenger cars registered were renewals. This is a much higher percentage than would be encountered in a normal year, because new car production had been suspended and the used-car market was also relatively inactive under war conditions. The number of new cars registered was naturally very small. Immigrations, on the other hand, constituted about 6½ percent of the vehicles registered. The number of returns, although relatively small, was much greater than the number of new cars. It seems probable that the number of returns, which are indicative of vehicles kept in storage throughout the registration year 1942, would have been greater in a war year than in a normal year.

The right-hand chart in figure 2 shows the same distribution for the passenger car sample drawn in 1944. In the 1944 study, cross-over renewals were identified separately from regular renewals. Of the vehicles registered, 88½ percent were regular renewals and 2½ percent were cross-overs. The number of new cars was much less than in 1943, because the rationed supply of 1942 models was running out. Returns were somewhat less; and it may be said that this result was unexpected, since it was thought that 1944 would show a considerable return of cars from wartime storage. The percentage of immigrations was somewhat greater, accounting for 7 percent of the vehicles registered.

Figure 3 shows the year-model distribution of private passenger cars registered in 1943. The dark bars represent a year-model distribution obtained in an earlier study of the occupations of passenger-car owners, in which a sample of between 15 and 20 percent of the registrations was taken. The light bars represent the results given by the 4-percent sample taken in this study. It will be observed that the only difference of more than negligi-

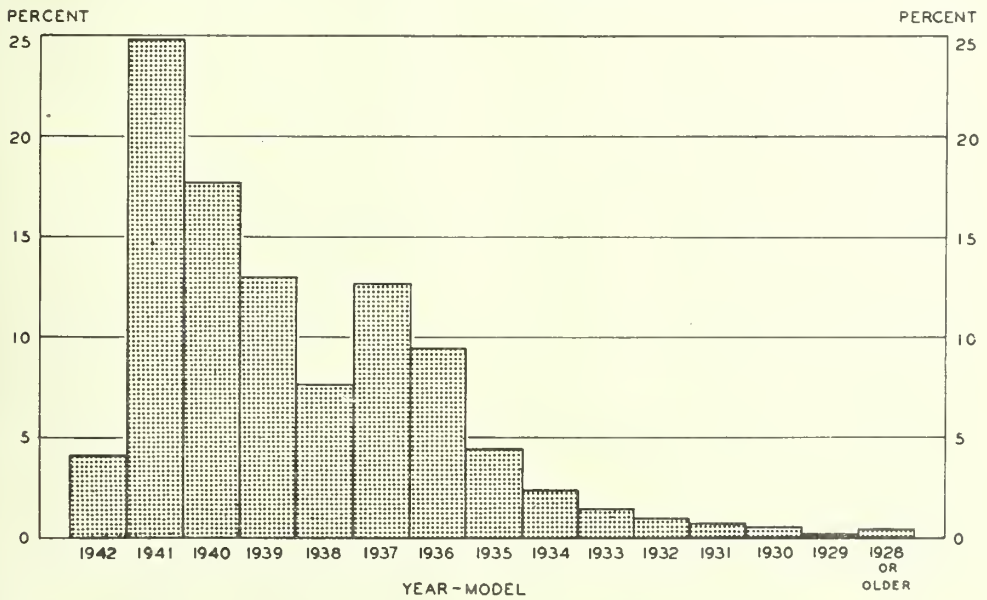

Figure 4.—Year-model distribution of private passenger cars in existence on March 1, 1943, which were subsequently registered for 1943.

ble size is found in the 1942 models. This disparity is accounted for by the fact that the occupational study was conducted with a cut-off date of June 30, 1943, while in the present study registrations of the entire year were sampled, so that all of the 1942 models were accounted for. This external check indicates that the sample size chosen was sufficient for an accurate representation of the year-model distribution.

Figure 4 shows the year-model distribution of private passenger cars in existence in the District of Columbia on March 1, 1943, which were subsequently registered for 1943. This figure serves to introduce the question of the determination of the number of vehicles in existence at the beginning or end of a given registration year. The drawing and identification of the sample served only to provide an estimate of the total number of vehicles that were registered in the year 1943. Since a certain number would be scrapped and others would be moved out of the District by their owners, the total number of vehicles registered would by no means represent the number existing at the end of the year.

An accurate estimate of the number of vehicles existing can be obtained from the registrations of a given year only by a throw-back to the beginning of the year. The process, as applied to the year 1943, is as follows: From the total number of vehicles registered in the year 1943 deduct the number of new vehicles registered and the number of duplicating immigrations; then add the returns of the year 1944, as representing 1943 withdrawals. The result is an estimate of the number of vehicles in existence in the District of Columbia at the beginning of the registration year 1943. Cross-over immigrations are not deducted, for these vehicles are either known or assumed to have been in the District at the beginning of the year.

The process described above is illustrated for private passenger cars in 1943, as follows:

Total number registered in 1943		114, 700
Less: New cars registered	700	
Duplicating immigrations	4, 400	
		−5, 100
Plus 1944 returns (representing 1943 withdrawals)		+1, 380
Total = Estimate of number of cars in existence at beginning of 1943		110, 980

In preparing the estimate of cars existing at the beginning of 1944, it was necessary to make an assumption regarding 1944 withdrawals, since no data on 1945 returns had been collected. A value equal to 1¼ times that of 1944 returns was selected, on the ground that there would be a somewhat greater accumulation of cars withheld from registration during 1944 than during 1943. The calculation for private passenger cars in 1944 is as follows:

Total number registered in 1944		98, 300
Less: New cars registered	360	
Duplicating immigrations	4, 630	
		−4, 990
Plus estimated 1944 withdrawals		+1, 725
Total = Estimate of number of cars in existence at beginning of 1944		95, 035

An important part of the study was the determination of the error involved in using the registration totals of a given year to represent the number of vehicles existing at the end of the year. Obviously, the sample estimate of cars existing at the beginning of 1943 should be compared with official total registrations for 1942; and similarly, cars existing at the beginning of 1944 should be compared with the official registrations for 1943. This comparison is given for passenger cars in figure 5. It will be observed that in both cases the official registration total was considerably in excess of the estimated number of cars existing. The divergence in 1942 was 22 percent and in 1943, 17 percent. Since registrations in the District of Columbia were declining rather rapidly during the war period, there is a likelihood that this discrepancy is greater than that which would be observed in a normal year. However, the sale of new cars induces scrappage, and in a year when such scrappage is of large magnitude, the discrepancy between registrations and cars existing might be rather high. In any event, the result of this sample study should serve as a warning of the danger of using registration totals as indicative of the number of vehicles existing at the end of the year.

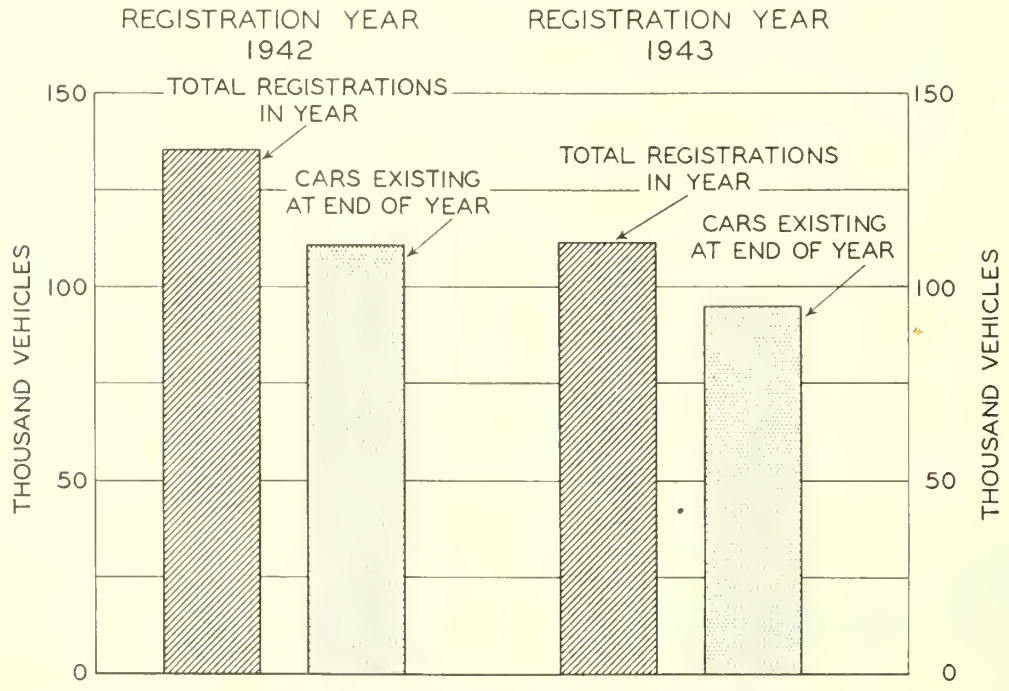

Figure 5.—Comparisons of official registrations of private passenger cars with estimated numbers of cars existing at the ends of corresponding years.

The severity of the discrepancy indicated in figure 5 is mitigated somewhat if a distinction is made between cars existing at the end of the expiring registration year and cars existing at the beginning of the new year. Every year there will be a number of cars that are not scrapped until the owner is confronted with the necessity of buying new plates for an unserviceable vehicle. Such cars may actually be driven on the last day of the expiring year. Statistically, there seems to be no advantage in making a distinction between the end of the old year and the beginning of the new; and it is customary to consider a vehicle that is scrapped without being registered for the new year as belonging to scrappage of the expiring year. On the other hand, since the magnitude of year-end scrappage is probably greater than proportional to total scrappage during a year, consideration of these vehicles as existing at the end of the old year would reduce the discrepancy indicated in figure 5, perhaps by an appreciable percentage. The registration figures, however, do not afford a means of estimating the number of vehicles involved.

It was also possible in this study to analyze the transition from the number of passenger cars existing at the beginning of the registration year 1943 to the number existing at the beginning of the registration year 1944. This transition, shown in the form of an accounting statement, is as follows:

Cars existing, March 1, 1943		110, 980
Changes during 1943 registration year:		
New cars registered	700	
Duplicating immigrations, 1943	4, 400	
Cross-over immigrations, 1943–44	2, 450	
Total vehicles added	7, 550	
Less scrappage and emigrations	−23, 495	
Net change in number of cars		−15, 945
Cars existing, March 1, 1944		95, 035

The process of accounting is evident: To the number of vehicles existing at the beginning

of 1943, add new cars registered in 1943, duplicating immigrations of 1943, and cross-over immigrations of the 1943–44 transition period; and deduct the number of vehicles which were scrapped or moved out of the State during the year. Actually, the latter is the residual figure, since its value can only be derived from the estimated cars existing at the beginnings of the two years, and the new cars and immigrations of the period between them. If the system of analysis herein described were adopted by all or a large number of States, the exchange of information among them would make possible an estimate of emigrations; and the number of vehicles scrapped would be derived as the residual figure.

The indicated volume of scrappage and emigrations, 23,495, seems rather high, particularly in view of the fact that scrappage is generally considered to have been at a minimum during the war. It must be remembered, however, that the population of the District of Columbia was rather fluid during the war years. In addition to military men and others moving out of the area, there was a steady drift of District of Columbia residents into new residential developments of the surburban sections of Maryland and Virginia.

In the 1944 study, other items of information were collected and analyzed, including the occupations of passenger-car owners, the geographical distribution of passenger cars registered, the weight distribution of all vehicle types, registration fees and personal property taxes paid, and the body-type distribution of trucks and tractor trucks. Space does not permit discussion of these parts of the analysis. In general, it was concluded that studies of this character could be made annually in all States at not too great expense, and that the information gained would be of substantial value to highway research and to the motor-vehicle industry.

THE TITLE STUDY

In the District of Columbia, certificates of title are numbered serially as issued, and the completed application forms are placed in a serial file, together with any documents auxiliary to the transaction. Such documents include (1) titles held by the next previous owner and surrendered by the new owner upon application for the new title, and (2) titles issued by other States and surrendered upon application for a title in the District of Columbia. The first group bear dated assignments of title, two in case of a consumer-to-dealer-to-consumer transaction and one in the case of a consumer-to-consumer sale. The second group bear similar assignments if the immigration involved a sale transaction. This method of filing results in a fully chronological file, any segment of which can be analyzed or sampled at any time. Those titles issued during the calendar year 1941 were selected for this study.

All vehicle types and classes filed separately in the title records were subjected to study; but the greatest attention was concentrated on passenger cars, which accounted for nearly 81,000 of the 88,600 titles issued in 1941.

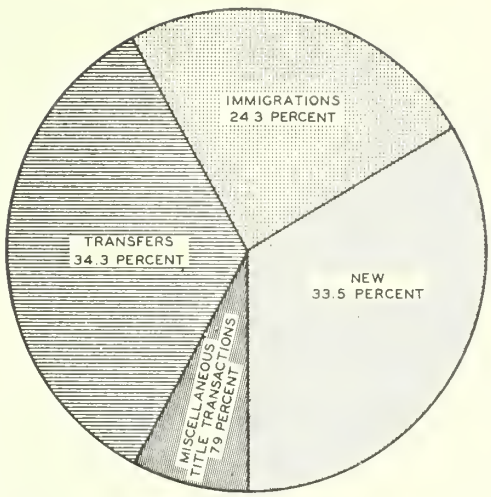

Figure 6.—Percentage distribution of private passenger-car title transactions in 1941.

Because of wide variation in the numbers of vehicles in the several classes to be sampled, the rate of periodic sampling varied from 1 in 50 for passenger cars to 1 in 3 for sightseeing vehicles.

The enumeration schedule was arranged so as to record the month of title issuance, year model of the vehicle, and the several kinds of titles, including new originals (issued for new vehicles), used originals (indicating an immigration), used transfers (issued upon transfer of ownership from one District of Columbia consumer-owner to another), and certain miscellaneous transactions, such as motor-number changes and titles issued to manufacturers and dealers, which were of little or no consequence in the study. A record was made also of the type of transaction, whether a dealer sale to consumer, a consumer-to-consumer sale, or an immigration not involving a sale transaction. Dates of title assignments were also taken off for those titles resulting from sales by District of Columbia dealers.

Figure 6 gives a summary of the results of the sampling of passenger-car titles. The miscellaneous transactions, in which there was no interest for this study, accounted for 7.9 percent of the total. Transfers of ownership accounted for the largest segment, 34.3 percent. The number of new cars titled was nearly as great, and immigrations accounted for 24.3 percent of the titles issued. The number of new car titles was 17.6 percent of total passenger-car registrations in 1941, immigrations were 12.8 percent, and transfers of ownership 18.1 percent. These figures illustrate the fact that a very large percentage of the cars registered in a normal year are involved in transactions of this sort.

Of the total passenger-car immigrations in 1941, estimated at 19,700, 57 percent were moved into the District of Columbia by the owners and 43 percent were brought in as a result of sale transactions. In 1941 many people were moving into the District because of National defense activities. In a normal year, or in an ordinary State, it is probable that sale transactions would account for a larger percentage of the immigrations.

A record was also made of the States from which the migrating vehicles came. Thirty-six States, including all of the 9 census regions, were represented in the sample total of 393 individuals. There is little doubt that, if the sample size had been sufficient, all of the 48 States would have been found present. However, the two neighboring States of Maryland and Virginia accounted for 68 percent of the total passenger-car immigrations to the District of Columbia in 1941.

Another interesting comparison is that between dealers' sales of used cars to consumers and consumer-to-consumer sales. Of the 724 sample individuals representing used-car sales, 23 percent were consumer-to-consumer sales, and 77 percent were dealer-to-consumer sales. It is probable that the percentage of consumer-to-consumer sales increased during the war years and is still high, because of the stringencies of the new-car market.

Although the 2-percent sample of 1941 passenger-car titles was inadequate to give a distribution of transactions by month of issue with a satisfactory degree of accuracy, the data were so analyzed; and the results displayed sufficient consistency and regularity of seasonal variation to justify their use in demonstrating the resources of the title file.

Figure 7 gives some of the results of this monthly analysis. Plotted values represent numbers of sample individuals. Maximum sales of new cars occurred in May, which was also the peak month for the Nation as a whole in 1941. The data are plotted by date of title assignment from dealer to purchaser, rather than by date of issue of the new title, thereby giving a more accurate representation of the time of sale. The tendency of used-car sales to follow the trend of new-car sales is plainly evident, although the peak in the month of May is decidedly less sharp.

The evidence of this figure seems to belie the often-quoted statement that two used cars must be sold for every new car. The data shown, however, do not include used-car sales by District of Columbia dealers to dealers and consumers in the suburban and rural areas surrounding the District. Furthermore, consumer-to-consumer sales and sales by dealers in other States (mainly Maryland and Virginia) are also omitted. If the Maryland and Virginia titles were analyzed for a common period with those of the District of Columbia, the characteristics of the used-car market in the District could be pretty well determined.

Figure 7 also portrays an estimate, in terms of sample individuals, of dealers' inventories of used cars at the end of each month, a quantity which it is necessary to have if it is desired to convert from cars existing to cars in use at any time. The following example will illustrate the method of arriving at these figures. If the dates of title assignments show that a car was sold to a dealer prior to December 31, 1940 and was sold by the dealer after that date, the car was obviously a part of dealers' inventories on that date. The used-car sales of each month (as given by date of title assignment from dealer to

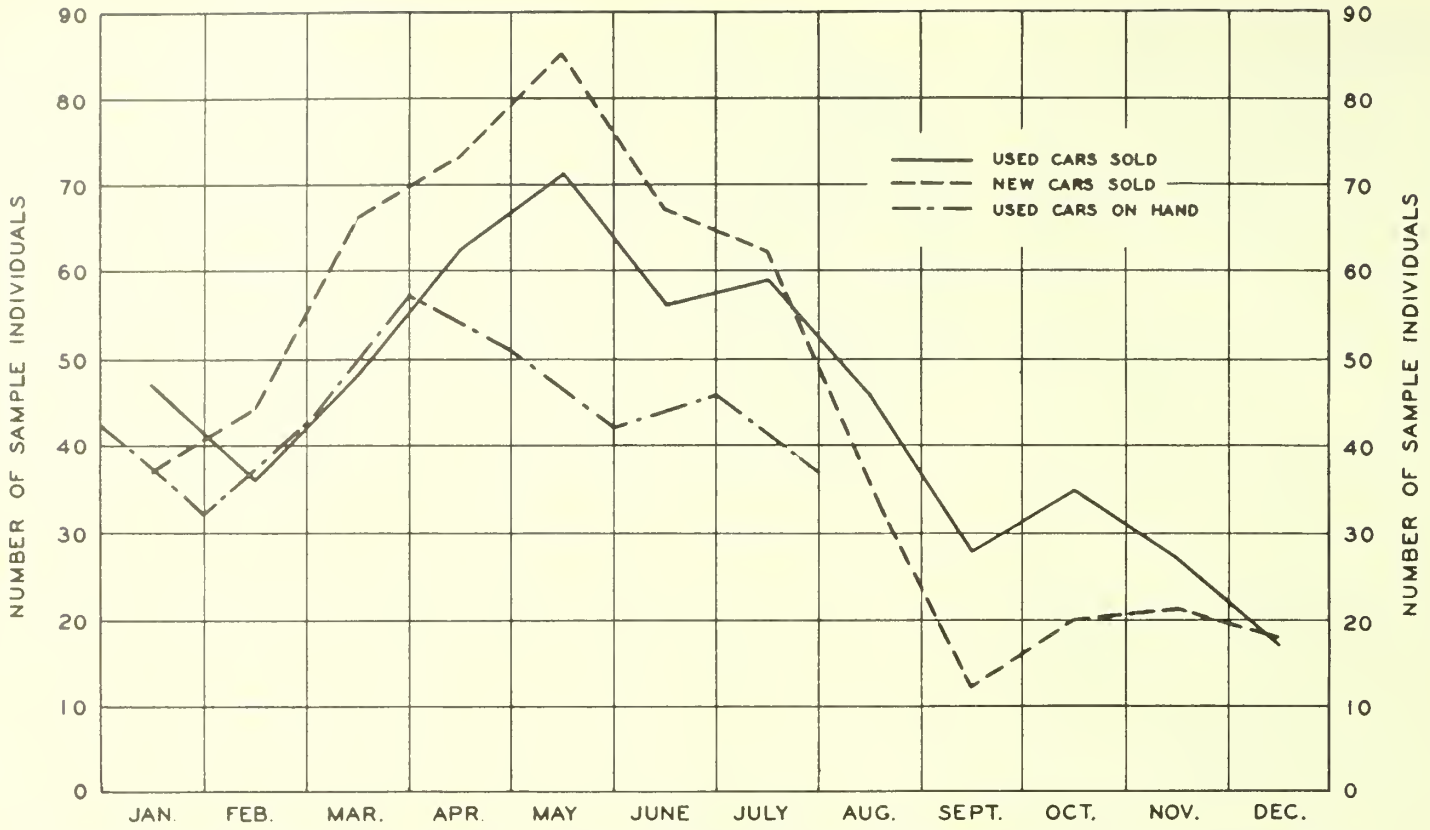

Figure 7.—Monthly distribution of dealers' sales of new and used cars, and used cars on hand at end of month, in 1941.

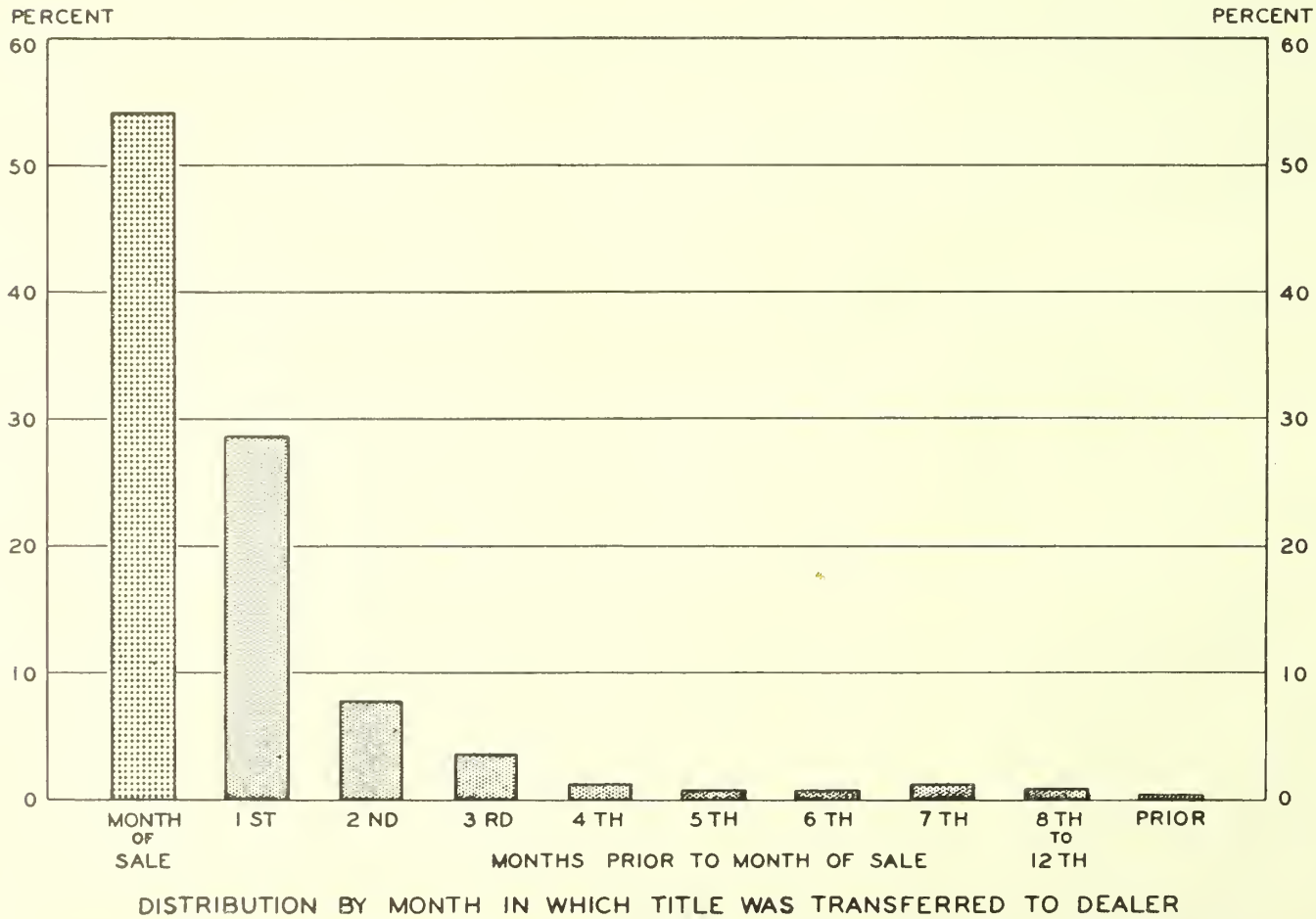

Figure 8.—Average percentage distribution of monthly sales of used passenger cars by dealers to consumers, by month in which title was transferred to dealer.

purchaser) were tabulated against month of purchase by the dealer, and the size of the dealers' inventories, or cars on hand, at the end of each month was obtained by deducting cumulative cars sold from cumulative cars purchased.

Dealers' inventories of used cars as obtained in this manner do not include cars destined to be sold to junkers rather than to consumers, or cars destined to be sold outside the District of Columbia, as for example, in the neighboring States of Maryland and Virginia. For this reason, the total given by expansion of the sample value of cars on hand would probably be materially less than the value given by a physical inventory of used cars in the hands of District of Columbia dealers. The quantity does have a definite and important significance, however—it may be defined as the effective used-car inventory of District of Columbia dealers with respect to the District of Columbia consumers' market.

The curve of dealers' inventories, or cars on hand, appears to follow a normal trend, having a pronounced maximum at the close of the registration year (March 31), and showing the next highest value on April 30, just preceding the month of maximum sales. It was impossible to carry this curve beyond the end of July, because of the absence of data regarding sales after December 31, 1941.

In order to develop an index of the rate of turn-over of used cars, the value of dealers' inventories, or cars on hand, was expressed in terms of days' supply on hand by referring to the average daily sales in the subsequent month. It was found that days' supply, computed in this manner, varied from 22 to 28 days.

The rate of turn-over of used cars in the District of Columbia was approximated by another means. Figure 8 gives the average percentage distribution of monthly sales of used cars by month in which the car was purchased. It will be observed that the vast majority of cars were sold either in the month in which they were purchased or in the first month thereafter. On the other hand, 5.3 percent of the 532 cars reported in the sample were sold after remaining four or more months in the hands of the dealer. The weighted average number of days in dealers' hands was 34.9. This quantity is comparable to days' supply, which was found to vary from 22 to 28 days. That the weighted average is materially greater than days' supply is due to the weighting computation, which magnifies the effect of the small number of cars that remained in the dealers' hands for several months. It is probable that the rate of turn-over of used cars in the District of Columbia, shown in this study to be in the neighborhood of 30 days, is materially shorter than in the average State, because of the effect of more leisurely ways of doing business in small cities and rural areas.

These examples are sufficient to illustrate the wealth of information regarding the characteristics of motor-vehicle ownership and the motor-vehicle market to be found in the files of certificates of title. Such material is perhaps of more consequence to the industry than to highway research, although it is believed that a knowledge of the dynamics of the motor-vehicle market is a valuable asset in highway research and planning, particularly with respect to the forecasting of highway revenue and of fluctuations in the volume of travel.

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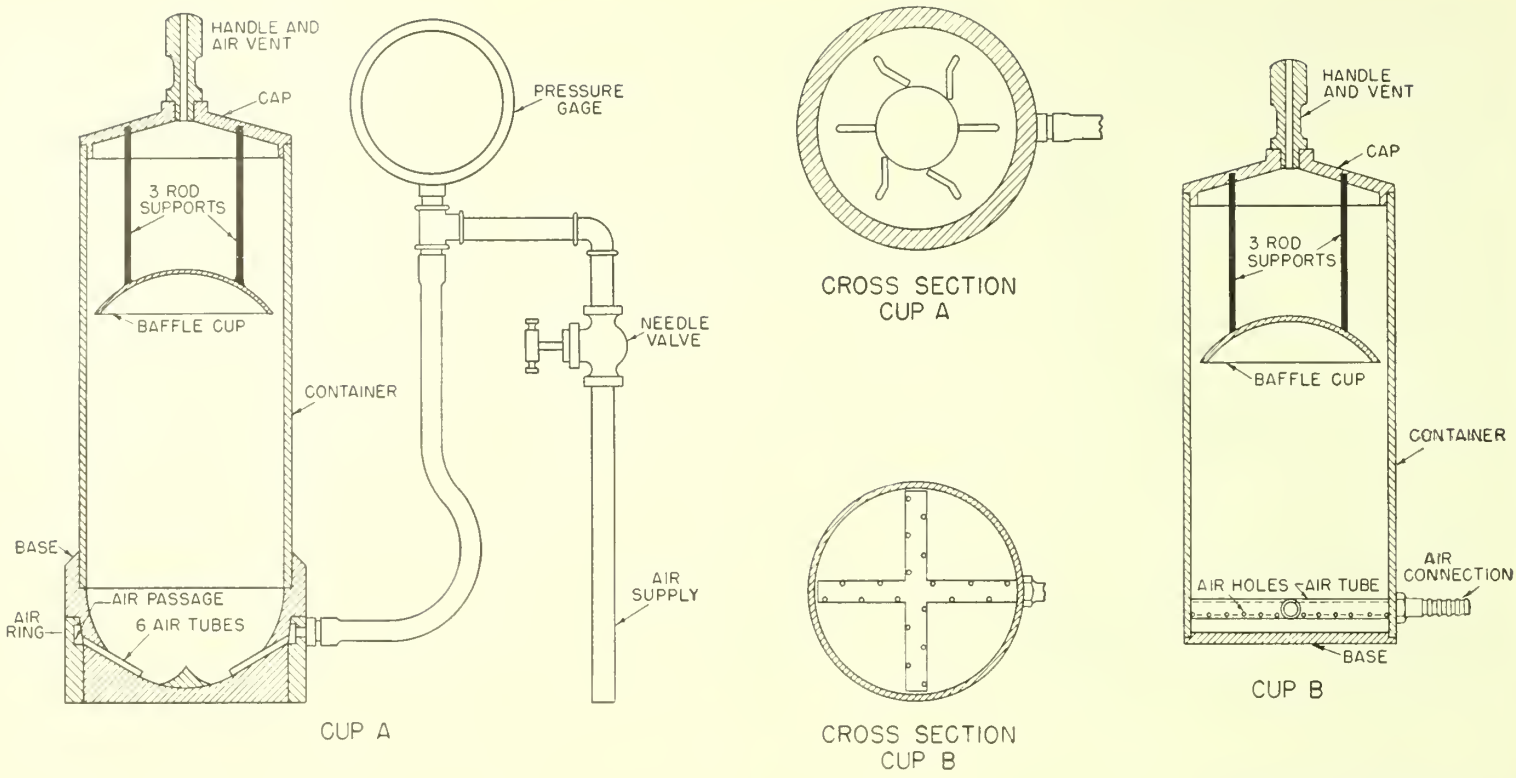

Figure 1.—The Wintermyer soil-dispersion cups.

A New Soil-Dispersing Apparatus For Mechanical Analysis of Soils

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by A. M. WINTERMYER,
Highway Engineer

The complete mechanical analysis of soils includes the hydrometer analysis of the fine-particle fraction of the sample, suspended in water. In the present standard method of preparing this suspension a mechanical stirring device is used to disperse the soil particles. This report describes a new apparatus utilizing air jets to stir the soil-water mixture. The air-jet cup provides more complete dispersion of the clay portion of plastic soils, yet its degrading action is no greater than that of the present standard stirrer. The new device can thus permit analysis more nearly representative of the actual gradations of the soils being tested.

THE ORIGINAL hydrometer method of mechanical analysis of soils was developed by Bouyoucos [1] in 1927. In this method of test a hydrometer is used to determine the percentage, by weight, of a sample of soil remaining in suspension in water after any given period of sedimentation. Since the velocity of sedimentation of soil particles of different sizes varies in accordance with Stokes' law, it is possible to compute the maximum grain size in suspension at the time any hydrometer reading is made. A series of such readings makes possible the determination of the gradation of the fraction of soil passing about the No. 200 sieve, which cannot be determined accurately by dry sieving methods. These data, combined with the sieve analysis of the washed material retained on the No 200 sieve, provide complete mechanical analysis of the soil.

A modification of the original Bouyoucos method was described in PUBLIC ROADS [2] in 1931. That modification, with some revisions, was adopted by the American Association of State Highway Officials as the standard method of mechanical analysis of soils.[3]

[1] *The hydrometer as a new and rapid method for determining the colloidal content of soils*, and *The hydrometer as a new method for the mechanical analysis of soils*, by G. J. Bouyoucos; Soil Science, vol. 23, No. 4, April 1927, p. 319 and vol. 23, No. 5, May 1927, p. 343.

[2] *Procedures for testing soils for the determination of the subgrade soil constants*, by A. M. Wintermyer, E. A. Willis, and R. C. Thoreen; and *Graphical solution of the data furnished by the hydrometer method of analysis*, by E. A. Willis, F. A. Robeson, and C. M. Johnston; PUBLIC ROADS, vol. 12, No. 8, October 1931.

[3] A. A. S. H. O. standard test method T 88–42, mechanical analysis of soils.

In the combined sieve and hydrometer method of mechanical analysis, it is necessary to use a mechanical device to disperse the soil thoroughly in water. The standard A. A. S. H. O. method specifies a high-speed stirring device with a metal paddle and a dispersion cup with metal baffles on its walls, similar in appearance to an electric milk-shaker. Various investigators have criticized the use of this apparatus on the ground that the agitation was of such a nature as to produce degradation [4] of the coarser soil particles. In fact, this criticism was responsible for the reduction of the dispersing time from 15 minutes as originally proposed to 1 minute as now specified by the standard A. A. S. H. O. test method. However, this modification has resulted in decreased efficiency for the dispersion of the clay fraction, especially in the more cohesive soils.

This report describes a new type of dispersing apparatus, utilizing the turbulence created by air jets to stir the soil-water mixtures, which has been developed to overcome the faults of the high-speed stirring device.

CONCLUSIONS

Comparative tests of a large number of soil samples, using the standard A. A. S. H. O. high-speed stirring apparatus and the new air-jet dispersing cup, are the basis for the following conclusions:

1. The degrading action of the air-jet dispersing cup with dispersing periods up to 20 minutes is no greater than that of the high-speed stirring device with a 1-minute dispersing period.

2. The use of the air-jet dispersing cup results in more efficient dispersion of the clay fraction of plastic soil samples than is obtained with the high-speed stirrer.

3. The air-jet dispersing cup, when used in accordance with the tentative procedure described in this report, will result in mechanical analyses more nearly representative of the actual gradations than those obtained by using the stirring device specified in the A. A. S. H. O. test method T 88–42.

DEVELOPMENT OF THE AIR-JET DISPERSING APPARATUS

Preliminary studies were conducted to determine the feasibility of using air jets for dispersing soil in water, the most effective shape for the dispersion cup, the effect of using a baffle plate, and the influence of such variables as air pressure, time of dispersion, volume of mixture to be dispersed, and method of treatment before dispersion. As a result of this exploratory work, an air-jet dispersing cup was designed which was believed to be suitable for the preparation of the soil-water suspensions required for the mechanical analysis of soils by the combined sieve and hydrometer method. A diagrammatic sketch of this apparatus, identified as cup *A*,[5] is shown in figure 1.

[4] As used in this article, degradation means the wearing down or breaking of soil particles into smaller sizes.

[5] Working drawings of the air-jet dispersion cups *A* and *B* described in this article may be obtained from the Public Roads Administration.

Table 1.—*Comparison of mechanical analyses using different methods of dispersion*

Sample No.	Plasticity index	Liquid limit	Dispersion method	Percentage of particles smaller than—					
				2.0 mm.	0.42 mm.	0.074 mm.	0.050 mm.	0.005 mm.	0.001 mm.
5	3	48	A. A. S. H. O.[1]	100	100	71	62	22	11
			Cup *A*[2]	100	95	68	57	21	11
6	62	88	A. A. S. H. O.	100	94	84	81	61	44
			Cup *A*	100	94	84	82	63	48
7	36	70	A. A. S. H. O.	100	99	95	94	62	26
			Cup *A*	100	99	95	95	60	27
8	7	46	A. A. S. H. O.	100	100	81	74	21	6
			Cup *A*	100	100	80	70	18	11
9	11	30	A. A. S. H. O.	100	100	88	82	40	23
			Cup *A*	100	99	86	81	38	21
10	11	28	A. A. S. H. O.	100	89	35	32	12	6
			Cup *A*	100	78	30	26	11	5
11	NP[3]	37	A. A. S. H. O.	100	98	68	54	6	2
			Cup *A*	100	90	50	40	8	5
12	40	67	A. A. S. H. O.	100	99	95	93	66	45
			Cup *A*	100	98	95	94	72	46
13	23	54	A. A. S. H. O.	100	100	84	75	48	21
			Cup *A*	100	100	79	71	41	26

[1] Tested in accordance with A. A. S. H. O. test method T 88-42.
[2] Average results obtained from tests in which soil was dispersed in cup *A* using pressures ranging from 2 to 20 psi, mixing time from 1 to 15 minutes, and various methods of pretreatment.
[3] Nonplastic.

Compressed air, after passing through a needle valve and pressure gage, enters an air ring encircling the base of the apparatus. Six radial tubes connected to the ring direct the compressed air against the specially shaped center of the bottom of the cup, vigorously agitating the soil-water mixture. After bubbling through the liquid, the air escapes through a vent in the handle of the cap which fits tightly over the top of the cup. A baffle suspended below the cap prevents the liquid from being thrown against the vent and blown out of the cup.

The simplified air-jet dispersion cup *B* was subsequently designed, and is also shown diagrammatically in figure 1. It differs from cup *A* primarily in the manner in which the compressed air is introduced. Cup *B* has four radial air tubes, parallel to and raised somewhat above the flat bottom of the cup. Compressed air is introduced at the outer end of one tube and, since the tubes are interconnected at the center, passes into the other three. Small apertures in the sides of the tubes jet the compressed air downward into the liquid. Cup *B* is much simpler to construct and, as will be shown later, is practically as efficient in dispersing soils as cup *A*.

Preliminary studies were made, using cup *A*, to establish a satisfactory procedure for using the air-jet device and to determine whether this apparatus would give results equal to or better than the standard A. A. S. H. O. high-speed stirrer. The results of tests on a group of representative soils having a wide range in physical properties are summarized in table 1. The values shown for the A. A. S. H. O. method of dispersion were obtained after dispersing the soil in accordance with A. A. S. H. O. method T88–42; those shown for cup *A* are average values for a number of tests in which the agitating air pressure was varied from 2 to 20 pounds per square inch, the mixing time was varied from 1 to 15 minutes, and various methods of pretreatment were employed.

The results of these preliminary tests gave the following indications:

1. The percentages of particles smaller than 0.42 mm. (No. 40 sieve) and 0.074 mm. (No. 200 sieve) obtained in the mechanical analyses using cup *A* were equal to or less than the corresponding percentages obtained by the A. A. S. H. O. method, indicating greater breakage and abrasion of the sand particles in the high-speed stirring device. This was more apparent in some soils than in others (see data for soil samples *5*, *10*, *11*, and *13* in table 1).

2. The degrading action of the stirring device was indicated in most cases by the increased percentages of material smaller than 0.05 mm. (silt and clay) although a reversal of this trend, caused by the more efficient dispersion of the clay and colloidal fractions in the air-jet device, began to be apparent in the analyses of some soils such as samples *6*, *7*, and *12*.

3. The percentages of particles smaller than 0.001 mm. obtained in the analyses using cup *A* were approximately equal to or greater than the corresponding percentages obtained by the A. A. S. H. O. method, indicating that greater dispersion was being obtained with the air-jet device for some soils and that the dispersing action of this new device was equal to that of the high-speed stirring apparatus for all soils tested.

These exploratory tests also showed that air pressure between 2 and 20 pounds per square inch could be used without appreciably affecting the particle-size distribution of the soils. In all tests subsequently reported, an air pressure of 20 pounds per square inch was used.

ABRASION CHARACTERISTICS

In order to obtain more conclusive data on the abrasion and dispersing characteristics of the two types of dispersing apparatus, an additional series of tests was run.

A quartz sand (Potomac River sand) and a commercial limestone sand were selected for the study of the abrasion characteristics of the two types of dispersing apparatus. Three samples were prepared from each sand, having maximum-size particles of 2.0 mm., 0.42 mm., and 0.074 mm., respectively. In this way, it was possible to study the effect on abrasion loss of maximum-particle size as well as the character of the sand grains.

The samples were quartered into 100-gram

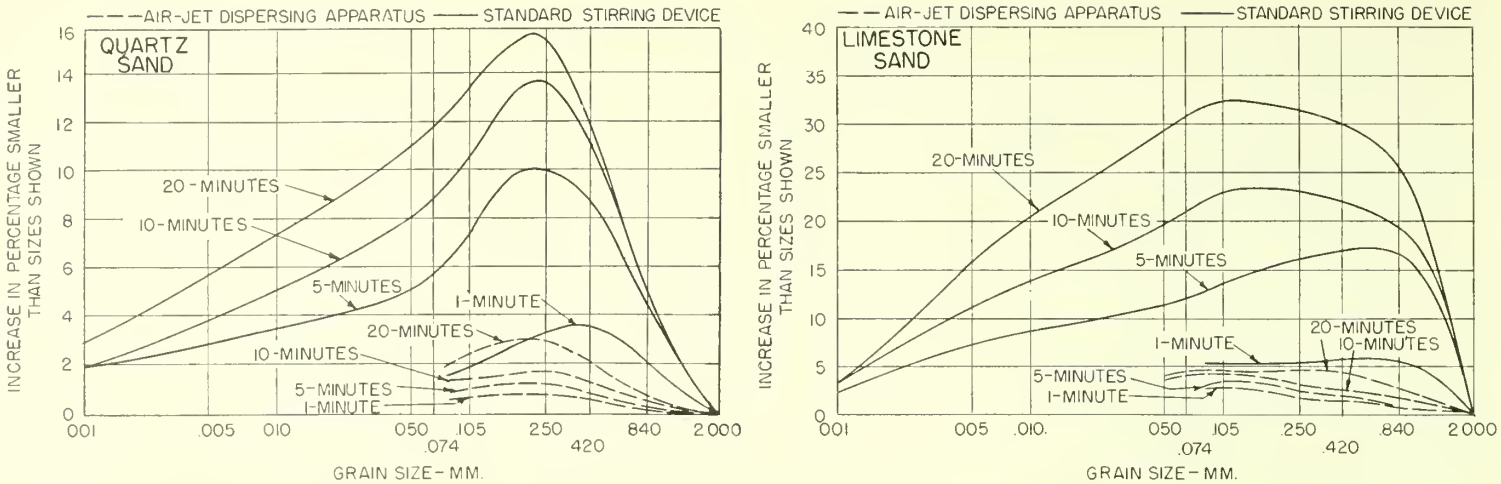

Figure 2.—Change in particle-size distribution due to abrasion of soil particles (maximum size 2.0 mm.).

individual test samples which were dispersed in each type of apparatus for 1, 5, 10, and 20 minutes. All tests were made in triplicate. The average test results are shown in table 2. An increase in abrasion of the mineral soil particles is indicated by a decrease in coarse sizes with a corresponding increase in fine sizes.

Table 2.—*Mechanical analyses of sands after various periods of dispersion*

Dispersion time and method	Percentage of particles smaller than—								
	2.0 mm.	0.84 mm.	0.42 mm.	0.25 mm.	0.105 mm.	0.074 mm.	0.050 mm.	0.005 mm.	0.001 mm.
QUARTZ SAND									
Undispersed	100	76	37	9	0	0	0	0	0
1 minute: A. A. S. H. O	100	78	41	12	2	2	----	----	----
Cup *A*	100	76	38	10	1	1	----	----	----
5 minutes: A. A. S. H. O	100	80	46	19	8	6	5	3	2
Cup *A*	100	76	38	10	1	1	----	----	----
10 minutes: A. A. S. H. O	100	81	48	23	11	9	8	4	2
Cup *A*	100	76	38	11	2	1	----	----	----
20 minutes: A. A. S. H. O	100	81	49	24	14	12	11	6	3
Cup *A*	100	77	39	12	3	2	----	----	----
Undispersed	--	--	100	75	17	0	0	0	0
1 minute: A. A. S. H. O	----	----	100	75	18	2	----	----	----
Cup *A*	----	----	100	75	17	2	----	----	----
5 minutes: A. A. S. H. O	----	----	100	76	19	3	3	2	2
Cup *A*	----	----	100	75	18	2	2	2	2
10 minutes: A. A. S. H. O	--	--	100	76	19	4	4	3	2
Cup *A*	----	----	100	75	18	3	2	2	2
20 minutes: A. A. S. H. O	--	----	100	77	21	5	5	3	2
Cup *A*	----	----	100	76	19	4	3	3	2
1 minute: A. A. S. H. O	--	----	----	----	----	100	76	14	6
Cup *A*	----	----	----	----	----	100	75	14	5
5 minutes: A. A. S. H. O	----	----	----	----	----	100	76	15	6
Cup *A*	----	----	----	----	----	100	76	14	6
10 minutes: A. A. S. H. O	----	---	----	----	----	100	77	16	7
Cup *A*	----	----	----	----	----	100	77	15	7
20 minutes: A. A. S. H. O	----	----	----	----	----	100	76	15	6
Cup *A*	----	----	----	----	----	100	74	16	6
LIMESTONE SAND									
Undispersed	100	48	23	15	5	0	0	0	0
1 minute: A. A. S. H. O	100	53	29	20	10	5	----	----	----
Cup *A*	100	48	24	17	8	3	----	----	----
5 minutes: A. A. S. H. O	100	65	40	30	19	13	11	7	2
Cup *A*	100	49	25	17	8	3	----	----	----
10 minutes: A. A. S. H. O	100	67	45	37	28	22	19	11	3
Cup *A*	100	49	26	18	9	4	----	----	----
20 minutes: A. A. S. H. O	100	73	53	46	37	33	29	16	3
Cup *A*	100	51	27	19	9	5	4	----	----
Undispersed	----	----	100	55	18	0	0	0	0
1 minute: A. A. S. H. O	----	----	100	63	24	6	----	----	----
Cup *A*	----	----	100	61	23	5	----	----	----
5 minutes: A. A. S. H. O	----	----	100	66	29	10	9	7	3
Cup *A*	----	----	100	63	25	6	6	4	3
10 minutes: A. A. S. H. O	----	----	100	69	32	13	12	9	3
Cup *A*	----	----	100	63	26	7	7	5	3
20 minutes: A. A. S. H. O	----	----	100	72	37	18	17	9	3
Cup *A*	----	----	100	65	28	9	8	7	3
1 minute: A. A. S. H. O	----	----	----	----	----	100	72	7	1
Cup *A*	----	----	----	----	----	100	71	7	2
5 minutes: A. A. S. H. O	----	----	----	----	----	100	79	8	3
Cup *A*	----	----	----	----	----	100	79	9	3
10 minutes: A. A. S. H. O	----	----	----	----	----	100	80	9	3
Cup *A*	----	----	----	----	----	100	80	9	3
20 minutes: A. A. S. H. O	----	----	----	----	----	100	80	9	4
Cup *A*	----	----	----	----	----	100	80	10	4

Figures 2 and 3 show graphically the results of the study of abrasion characteristics of the two types of dispersing apparatus. In these figures the increases in the percentage smaller than any given size (differences between the results of the analyses before and after dispersion) are plotted against the respective grain sizes produced by the two types of apparatus in 1-, 5-, 10-, and 20-minute dispersing periods.

A study of these comparative test data shows that:

1. The abrasion losses are greater for samples dispersed in the standard high-speed stirring device than for similar samples dispersed for an equal length of time in the air-jet dispersion cup.

2. Degradation of the sand particles increases with increased time of dispersion in both types of apparatus. The increase in degradation with dispersion time is much greater in the stirring device than in the air-jet cup.

3. The abrasion losses in the standard stirrer are much greater in samples containing particles having a maximum grain size of 2.0 mm. than in samples having maximum grain sizes of 0.42 mm. or 0.074 mm.

4. The degradation of the sand samples containing particles with a maximum size of 2.0 mm. when dispersed for 20 minutes in cup *A* was less than that which occurred when duplicate samples were dispersed for 1 minute in the standard stirring device.

5. The amount of abrasion occurring in the samples having a maximum particle size of 0.074 mm. was negligible.

6. The mineral composition of the sand grains appears to be a factor influencing the degradation. In the tests reported here, the quartz sand was more resistant to fracture and abrasion than the limestone sand. However, it is not known to what extent the shape and hardness of the mineral particles affect the abrasion loss, since these factors were not studied separately during this investigation.

It appears then, that for sandy soils the use of the air-jet type of dispersion cup will result in lower abrasion losses than those which are to be expected when the standard A. A. S. H. O. dispersing procedure is fol-

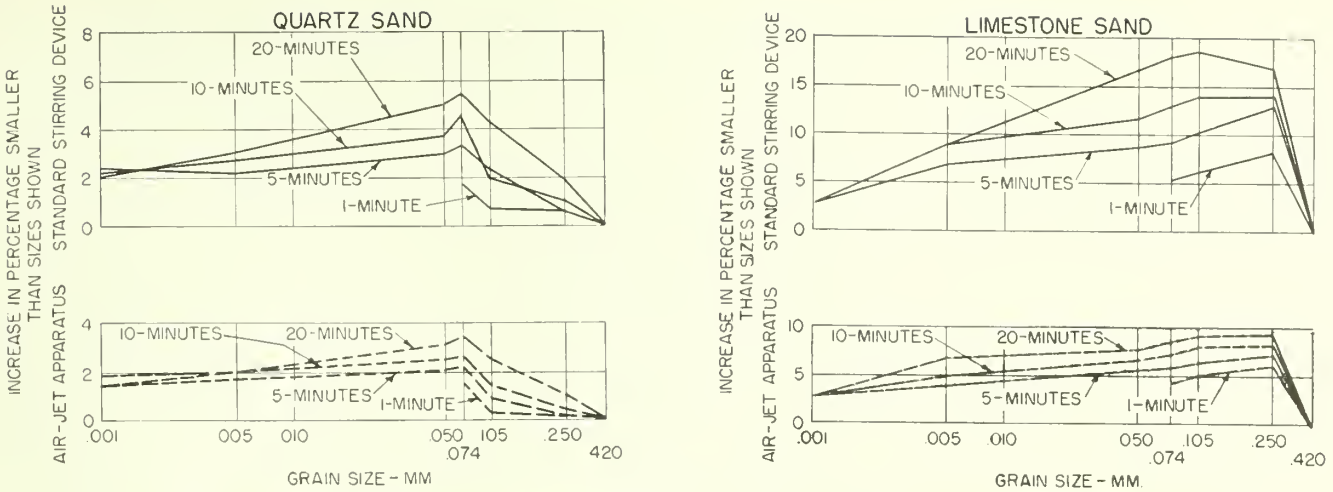

Figure 3.—Change in particle-size distribution due to abrasion of soil particles (maximum size 0.42 mm.).

lowed. In this connection, it was indicated that degradation could be materially reduced if the standard procedure required that the combined sieve and hydrometer analysis be made on the fraction passing the No. 40 sieve (0.42 mm.) instead of on the fraction passing the No. 10 sieve (2.0 mm.).

DISPERSING CHARACTERISTICS

Two clay soils were selected to study the dispersing characteristics of the two types of agitating devices. Test samples were prepared in a manner similar to that used for the two sands except that 50-gram instead of 100-gram individual test samples were used. Tests were made in each device for dispersing time intervals of 1, 5, 10, and 20 minutes and for test samples having maximum particle sizes of 2.0 mm., 0.42 mm., and 0.074 mm. In all tests in this series, the samples were soaked in water for 18 hours prior to dispersion. The results of tests are shown in table 3. Each value is the average of the results obtained on three samples.

A study of the test data for the Coastal Plain clay soil shows that for samples having maximum particle sizes of 2.0 mm. and 0.42 mm. there is very little difference in the dispersing characteristics of the two types of dispersing devices as measured by the percentages smaller than 0.005 mm. and 0.001 mm. About the same degree of dispersion was obtained in both devices with the 1-minute mixing period as with the 20-minute period. For the samples passing the No. 200 sieve, however, the percentage of colloidal material increased very considerably with increased time of dispersion. The air-jet dispersion was more effective at all dispersion periods.

The study of the test data for the Piedmont Plateau region clay soil indicates that the dispersing time is an important factor to be considered for this type of cohesive soil. The percentage of particles smaller than 0.001 mm. increased with an increase in the time of dispersion. The greatest change occurred between the 1- and 5-minute periods. Also, there was a marked difference in the dispersing characteristics of the two types of dispersion cups. The degree of dispersion at the end of 5 minutes for the air-jet device was greater than that found at the end of 20 minutes for the standard stirring apparatus.

It appears from the study of these data that the dispersing time for cohesive soils should be increased over the 1-minute period specified in the A. A. S. H. O. method in order

Table 3.—*Mechanical analyses of clay soils after various periods of dispersion*

Dispersion time and method	Percentage of particles smaller than—					
	2.0 mm.	0.42 mm.	0.074 mm.	0.050 mm.	0.005 mm.	0.001 mm.
COASTAL PLAIN SOIL [1]						
1 minute: A. A. S. H. O.	100	96	67	63	42	31
Cup *A*	100	96	67	63	41	32
5 minutes: A. A. S. H. O.	100	96	68	64	44	31
Cup *A*	100	96	68	64	44	32
10 minutes: A. A. S. H. O.	100	96	69	64	42	32
Cup *A*	100	96	67	63	43	32
20 minutes: A. A. S. H. O.	100	96	69	64	44	32
Cup *A*	100	96	68	65	45	34
1 minute: A. A. S. H. O.	.....	100	66	62	42	30
Cup *A*	.....	100	66	62	42	30
5 minutes: A. A. S. H. O.	.....	100	66	62	42	31
Cup *A*	.....	100	67	62	42	30
10 minutes: A. A. S. H. O.	.....	100	67	62	42	31
Cup *A*	.....	100	66	62	42	32
20 minutes: A. A. S. H. O.	.....	100	67	62	42	31
Cup *A*	.....	100	67	62	42	32
1 minute: A. A. S. H. O.	.....	.....	100	94	54	31
Cup *A*	.....	.....	100	94	58	36
5 minutes: A. A. S. H. O.	.....	.....	100	94	60	38
Cup *A*	.....	.....	100	95	61	42
10 minutes: A. A. S. H. O.	.....	.....	100	94	60	40
Cup *A*	.....	.....	100	94	62	44
20 minutes: A. A. S. H. O.	.....	.....	100	94	61	42
Cup *A*	.....	.....	100	92	61	45
PIEDMONT PLATEAU SOIL [2]						
1 minute: A. A. S. H. O.	100	91	82	79	59	30
Cup *A*	100	91	81	79	57	34
5 minutes: A. A. S. H. O.	100	93	83	80	62	42
Cup *A*	100	91	82	79	63	50
10 minutes: A. A. S. H. O.	100	92	84	82	63	43
Cup *A*	100	92	82	81	64	53
20 minutes: A. A. S. H. O.	100	94	85	83	64	48
Cup *A*	100	92	83	80	65	55
1 minute: A. A. S. H. O.	.....	100	87	85	60	30
Cup *A*	.....	100	87	84	59	32
5 minutes: A. A. S. H. O.	.....	100	88	85	65	41
Cup *A*	.....	100	87	84	66	50
10 minutes: A. A. S. H. O.	.....	100	88	85	65	44
Cup *A*	.....	100	88	86	68	54
20 minutes: A. A. S. H. O.	.....	100	88	85	66	46
Cup *A*	.....	100	88	86	67	54
1 minute: A. A. S. H. O.	.....	.....	100	96	64	34
Cup *A*	.....	.....	100	95	68	37
5 minutes: A. A. S. H. O.	.....	.....	100	96	67	43
Cup *A*	.....	.....	100	96	68	52
10 minutes: A. A. S. H. O.	.....	.....	100	96	67	46
Cup *A*	.....	.....	100	96	68	53
20 minutes: A. A. S. H. O.	.....	.....	100	96	66	50
Cup *A*	.....	.....	100	96	66	57

[1] A reddish-brown heavy clay soil derived from the weathering of unconsolidated sediments of the Coastal Plain region, with liquid limit of 50, plasticity index of 28, and specific gravity of 2.73.

[2] A yellow tenaceous residual clay derived from the weathering of diabasic rocks of the Piedmont Plateau region, with liquid limit of 76, plasticity index of 49, and specific gravity of 2.79.

TENTATIVE PROCEDURE FOR USING AIR-JET DISPERSION APPARATUS

The representative soil sample, prepared according to the A. A. S. H. O. standard method T 87–42, shall be weighed and placed in a 250-ml. beaker. Then about 150 ml. of distilled water shall be added and the soil–water mixture stirred until the soil is thoroughly wetted. This mixture shall be allowed to soak for at least 1 hour. It is recommended that plastic clay soils difficult to disperse be permitted to soak overnight, to facilitate the dispersion of the soil colloids.

The air-jet dispersion apparatus shall be assembled without the cover cap in place. The needle valve controlling the line pressure shall be opened until the pressure gage indicates 1 pound per square inch air pressure. This initial air pressure is required to prevent the soil–water mixture from entering the air-jet tubes when the mixture is transferred to the dispersion cup. After the apparatus is adjusted, the soil–water mixture shall be transferred from the beaker to the dispersion cup, using a wash bottle to assist in the transfer operation. The deflocculating agent as specified in A. A. S. H. O. method T 88–42 shall then be added.

The volume of the soil–water mixture in the dispersion cup shall not exceed 250 ml. The cover and affixed baffle plate shall be placed upon the dispersion cup and the needle valve opened until the pressure gage reads 20 pounds per square inch. The soil–water mixture shall be dispersed for 5, 10, or 15 minutes depending upon the plasticity index of the soil: Soils with a plasticity index of 5 or less shall be dispersed for 5 minutes; soils with a plasticity index between 6 and 20 for 10 minutes; and soils with a plasticity index greater than 20 for 15 minutes. Soils containing large percentages of mica need be dispersed for 1 minute only.

At the end of the dispersion period, the needle valve shall be closed until the pressure gage indicates 1 pound per square inch. The cover shall be removed and all adhering soil particles washed back into the dispersion cup. The soil–water suspension shall then be washed into the 1,000-ml. glass graduate and the needle valve closed. The remainder of the test procedure follows A. A. S. H. O. method T 88–42 for the mechanical analysis of soils.

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to insure adequate dispersion of the clay fraction. Since the cohesion is related to the plasticity, the time of dispersion should logically be varied with the plasticity index of the soil; that is, a short dispersion period should be used for soils with low plasticity indexes, an intermediate period for soils with moderate plasticity indexes, and longer dispersion periods for soils having high plasticity indexes.

USING THE AIR-JET APPARATUS

A tentative method of preparing soil-water suspensions for the mechanical analysis of soils by the hydrometer method using the new type of air-jet dispersion cup has been prepared, based on the comparative testing of a large number of soils. The results of tests on a few of the more representative soils are used in this report to indicate the relative performance of the air-jet dispersion cup as compared with the standard stirring device.

The method for the preparation of soils for test and the determination of particle-size distribution from the soil-water suspension by the soil hydrometer is the same as that designated in the A. A. S. H. O. standard methods T 87–42 and T 88–42. The only changes in the procedure are those required for using the air-jet dispersing cup instead of the standard high-speed stirrer and the use of different dispersion-time periods based upon the plasticity index of the soils. The tentative procedure for using the air-jet dispersion cup is presented elsewhere on this page.

DISPERSION CHARACTERISTICS OF REPRESENTATIVE SOILS

A series of representative soils were tested using the standard stirrer and the two air-jet dispersing cups (*A* and *B*) for the preparation of the soil–water suspensions. The soil samples dispersed by the standard stirring device were tested in accordance with the standard A. A. S. H. O. method T 88–42. The samples dispersed by the two types of air-jet dispersing cup were tested in accordance with the tentative procedure except that all soils were soaked for 18 hours prior to dispersion. The results of tests on this series of representative soils are given in table 4.

(*Continued on page 108*)

Table 4.—*Comparative mechanical analyses using three dispersion devices*

Soil						Dispersion method	Percentage of particles smaller than—					
Sample No.	Type	Derivation	Source	Liquid limit	Plasticity index		2.0 mm.	0.42 mm.	0.074 mm.	0.050 mm.	0.005 mm.	0.001 mm.
21	Plastic clay	Weathering of diabasic rocks.	Piedmont Plateau of Virginia.	88	62	A. A. S. H. O.	100	93	83	80	58	32
						Cup *A*	100	93	84	82	65	52
						Cup *B*	100	93	83	80	61	51
22	Clay	Unconsolidated sediments.	Coastal Plain of Virginia.	67	40	A. A. S. H. O.	100	99	95	93	70	46
						Cup *A*	100	99	96	95	75	50
						Cup *B*	100	98	95	93	70	48
23	do	Marl or chalk deposits.	Coastal Plain of Texas.	54	32	A. A. S. H. O.	100	100	79	72	35	13
						Cup *A*	100	100	79	72	44	32
						Cup *B*	100	100	79	71	42	29
24	do	Weathering of glacial till.	Minnesota	51	21	A. A. S. H. O.	100	94	76	70	16	6
						Cup *A*	100	95	83	76	33	18
						Cup *B*	100	95	73	66	30	16
25	do	Loess	Mississippi	42	17	A. A. S. H. O.	100	100	100	98	34	7
						Cup *A*	100	100	100	98	48	19
						Cup *B*	100	100	100	98	44	16
26	do	Weathering of glacial till.	Wisconsin	31	15	A. A. S. H. O.	100	93	73	68	30	10
						Cup *A*	100	93	73	70	40	22
						Cup *B*	100	93	73	68	40	21
27	Silt	Weathering of loess.	Iowa	33	7	A. A. S. H. O.	100	100	99	86	11	2
						Cup *A*	100	100	100	88	17	7
						Cup *B*	100	100	100	88	14	5
28	Muck	Tidal flats	Virginia	35	7	A. A. S. H. O.	100	99	89	80	16	5
						Cup *A*	100	100	90	82	26	9
						Cup *B*	100	99	90	81	27	9
29	Sand-clay		Coastal Plain of Alabama.	23	4	A. A. S. H. O.	100	83	22	21	18	14
						Cup *A*	100	84	25	22	19	17
						Cup *B*	100	84	23	22	19	17
30	Sand	Weathering of granitic rocks.	Piedmont Plateau of Virginia.	37	NP [1]	A. A. S. H. O.	100	98	68	54	6	2
						Cup *A*	100	95	57	47	8	6
						Cup *B*	100	94	49	41	8	4
31	do	do	do	40	NP	A. A. S. H. O.	100	94	36	28	6	2
						Cup *A*	100	80	30	25	6	3
						Cup *B*	100	82	36	29	7	5
32	Sand [2]		Coastal Plain of South Carolina.	NP	NP	A. A. S. H. O.	100	75	3	3	2	1
						Cup *A*	100	78	4	4	3	2
						Cup *B*	100	75	3	3	2	2

[1] Nonplastic.

[2] Contains some shell fragments.

The Federal-aid Highway Act of 1948

[Public Law 834—80th Congress]

[Chapter 732—2d Session]

[H. R. 5888]

AN ACT

To amend and supplement the Federal-aid Road Act approved July 11, 1916 (39 Stat. 355), as amended and supplemented, to authorize appropriations for continuing the construction of highways, and for other purposes.

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That for the purpose of carrying out the provisions of the Federal-Aid Road Act approved July 11, 1916 (39 Stat. 355), and all Acts amendatory thereof and supplementary thereto, and for continuing the construction and reconstruction of highways in accordance with the provisions of the Federal-aid Highway Act of 1944 approved December 20, 1944 (58 Stat. 838), there is hereby authorized to be appropriated the sum of $450,000,000 for the fiscal year ending June 30, 1950, and a like sum for the fiscal year ending June 30, 1951.

The sum herein authorized for each fiscal year shall be available for expenditure as follows:

Forty-five per centum for projects on the Federal-aid highway system.

Thirty per centum for projects as set forth in paragraph (b) of section 3 of the Federal-aid Highway Act of 1944 (58 Stat. 838), except that for the purposes of this Act and all subsequent Acts continuing the postwar construction and reconstruction of highways in accordance with the provisions of the Federal-aid Highway Act of 1944, (1) the term "secondary and feeder roads" and the term "principal secondary and feeder roads", wherever used in the Federal-aid Highway Act of 1944, shall include county and township roads; and (2) in selecting county and township roads on which funds are to be expended, the State highway departments shall cooperate with township trustees and other appropriate local road officials; and

Twenty-five per centum for projects on the Federal-aid highway system in urban areas.

The said sums, respectively, for any fiscal year, shall be apportioned among the several States in the manner now provided by law and in accordance with the formulas set forth in section 4 of the Federal-aid Highway Act of 1944 approved December 20, 1944: *Provided*, That the authorization for the fiscal year ending 1950 shall be apportioned among the States as soon as practicable after July 1, 1948, but not later than September 1, 1948.

Any sums apportioned to any State under the provisions of this section shall be available for expenditure in that State for two fiscal years after the close of the fiscal year for which such sums are authorized, and any sums apportioned to any State under section 4 of the Federal-aid Highway Act of 1944, approved December 20, 1944, shall be available for expenditure in that State for three fiscal years after the close of the fiscal year for which such sums are authorized and any amount so apportioned remaining unexpended at the end of such period shall lapse: *Provided*, That such funds for any fiscal year, including any funds authorized to be appropriated under this Act, shall be deemed to have been expended if a sum equal to the total of the sums apportioned to the State for such fiscal year is covered by formal agreements with the Commissioner of Public Roads for the improvement of specific projects as provided by this Act.

Sec. 2. The Commissioner of Public Roads is hereby directed to cooperate with the State highway departments in a study of the status of improvement of the National System of Interstate Highways, designated in accordance with the provisions of section 7 of the Federal-aid Highway Act of 1944; to invite the cooperation and suggestions of the Secretary of Defense and the National Security Resources Board as to their indicated or potential needs for improved highways for the national defense; and to supplement, not later than April 1, 1949, the report dated February 1, 1941, entitled "Highways for the National Defense" (Seventy-seventh Congress, first session), to reflect current conditions and deficiencies.

Sec. 3. (a) For the purpose of carrying out the provisions of section 23 of the Federal Highway Act (42 Stat. 218), as amended and supplemented, there is hereby authorized to be appropriated (1) for forest highways the sum of $20,000,000 for the fiscal year ending June 30, 1950, and a like sum for the fiscal year ending June 30, 1951, subject to the provision of section 9 of the Federal-aid Highway Act of 1944 respecting the apportionment for forest highways in Alaska; and (2) for forest development roads and trails the sum of $17,500,000 for the fiscal year ending June 30, 1950, and a like sum for the fiscal year ending June 30, 1951: *Provided*, That immediately upon the passage of this Act the appropriation herein authorized for forest highways for the fiscal year ending June 30, 1950, shall be apportioned by the Federal Works Administrator for expenditure in the several States, Alaska, and Puerto Rico, according to the area and value of the land owned by the Government within the national forests therein which the Secretary of Agriculture is hereby directed to determine and certify to him from such information, sources, and departments as the Secretary of Agriculture may deem most accurate, and hereafter, on or before January 1 next preceding the commencement of each succeeding fiscal year the Federal Works Administrator shall make like apportionment of the appropriation authorized for such fiscal year: *Provided further*, That the Commissioner of Public Roads may incur obligations, approve projects, and enter into contracts under the apportionment of such authorizations, and his action in so doing shall be deemed a contractual obligation of the Federal Government for the payment of the cost thereof: *Provided further*, That the appropriations made pursuant to authorizations heretofore, herein, and hereafter enacted for forest highways shall be considered available to the Commissioner of Public Roads for the purpose of discharging the obligations created hereunder in any State or Territory: *Provided further*, That the total expenditures on account of any State or Territory shall at no time exceed its authorized apportionment: *Provided further*, That appropriations for forest highways shall be administered in conformity with regulations jointly approved by the Federal Works Administrator and the Secretary of Agriculture: *Provided further*, That the Commissioner of Public Roads shall transfer to the Chief of the Forest Service from appropriations for forest highways such amounts as may be needed to cover necessary administrative expenses of the Forest Service in connection with the forest-highway program.

(b) The authorization in section 9 of the Federal-aid Highway Act of 1944 for forest highways for the fiscal year ending June 30, 1948, is hereby canceled.

(c) Hereafter, construction work on forest-development roads and trails, pursuant to the provisions of section 23 of the Federal Highway Act of November 9, 1921, as amended and supplemented, estimated to cost $10,000 or more per mile, exclusive of bridges, shall be advertised and let to contract. If such estimated cost is less than $10,000 per mile, or if, after proper advertising, no acceptable bid is received, or the bids are deemed excessive, the work may be done by the Secretary of Agriculture on his own account.

Sec. 4. (a) For the construction, reconstruction, improvement, and maintenance of roads and trails, inclusive of necessary bridges, in national parks, monuments, and other areas administered by the National Park Service, including areas authorized to be established as national parks and monuments, and national park and monument approach roads authorized by the Act of January 31, 1931 (46 Stat. 1053), as amended, there is hereby authorized to be appropriated the sum of $10,000,000 for the fiscal year ending June 30, 1950, and a like sum for the fiscal year ending June 30, 1951.

(b) For the construction and maintenance of parkways, to give access to national parks and national monuments, or to become connecting sections of a national parkway plan, over lands to which title has been transferred to the United States by the States or by private individuals, there is hereby authorized to be appropriated the sum of $10,000,000 for the fiscal year ending June 30, 1950, and a like sum for the fiscal year ending June 30, 1951.

(c) For the construction, improvement, and maintenance of Indian reservation roads and bridges and roads and bridges to provide access to Indian reservations and Indian lands under the provisions of the Act approved May 26, 1928 (45 Stat. 750), there is hereby authorized to be appropriated the sum of $6,000,000 for the fiscal year ending June 30, 1950, and a like sum for the fiscal year ending June 30, 1951: *Provided*, That the location, type, and design of all roads and bridges constructed shall be approved by the Public Roads Administration before any expenditures are made thereon, and all such construction shall be under the general supervision of the Public Roads Administration.

SEC. 5. All provisions of the Federal-aid Highway Act of 1944, approved December 20, 1944 (58 Stat. 838), not inconsistent with this Act, shall remain in full force and effect.

SEC. 6. The first paragraph of section 21 of the Federal Highway Act, approved November 9, 1921 (23 U. S. C. 21), is hereby amended to read as follows:

"That so much, not to exceed 3¾ per centum, of all moneys appropriated or authorized to be appropriated for expenditure under the provisions of this Act, as the Federal Works Administrator may deem necessary for administering the provisions of this Act and for carrying on necessary highway research and investigational studies independently or in cooperation with the State highway departments and other research agencies, and for publishing the results thereof, shall be deducted therefrom for such purposes when the apportionment is made and the amount so deducted shall be available until expended from appropriations made under the provisions of this Act: *Provided*, That should the apportionment of the amounts authorized for the third postwar fiscal year be made in accordance with section 4 of the Federal-aid Highway Act of 1944 before the approval of this Act, a revised apportionment may be made and the increased amount authorized by this section deducted for administration, research, and investigational studies."

SEC. 7. This Act may be cited as the "Federal-aid Highway Act of 1948".

Approved June 29, 1948.

(*Continued from page 106*)

The data presented are representative of a large number of analyses made on soils ranging from nonplastic sands through heavy plastic clays. The tests show that the use of the air-jet dispersion cup in accordance with the tentative procedure recommended does not degrade the soil sample more than the 1-minute dispersion called for in the standard A. A. S. H. O. method T 88–42, and that more efficient dispersion of the clay fraction is accomplished.

There appears to be very little difference in the dispersing efficiency of the two types of air-jet dispersing cups. Cup *A* shows slightly better dispersing characteristics than the simplified design, cup *B*, for the heavier soils shown in table 4.

In a few instances, in some of the analyses not reported in detail in this report, the degree of dispersion obtained with cup *B* was slightly less than that obtained with the standard stirring device. This was not the case with cup *A*. In every instance the degree of dispersion obtained with cup *A* was greater than that obtained with the standard disperser.

SUMMARY

The test data obtained led to the following general conclusions:

1. The use of the high-speed stirring device specified in A. A. S. H. O. method T88–42 for the preparation of soil-water suspensions for the combined sieve and hydrometer analysis of soils results in degradation of the sand particles. This action is most pronounced in the fraction between the No. 10 (2.0 mm.) and the No. 40 (0.42 mm.) sieves. The hardness of the sand grains is a factor in the amount of degradation that occurs.

2. The degrading action of the air-jet dispersing cup with dispersing periods up to 20 minutes is no greater than that of the high-speed stirring device with a 1-minute dispersing period.

3. The use of the air-jet dispersing cup results in more efficient dispersion of the clay fraction of plastic soil samples than is obtained with the high-speed stirrer.

4. The efficiency of the original air-jet dispersion apparatus, cup *A*, is slightly greater than that found for the simplified design, cup *B*. However, it is questionable whether the increased efficiency would justify the additional cost of cup *A*.

5. The maintenance costs for the air-jet type dispersion cup are considerably less than for the standard stirrer since it contains no moving parts.

6. The use of the air-jet dispersing cup, either type *A* or type *B*, in accordance with the tentative procedure contained in this report in which the dispersing time is varied with the plasticity index, will result in mechanical analyses more nearly representative of the actual gradations than those obtained by using the stirring device called for in the A. A. S. H. O. method for mechanical analysis of soils.

U. S. GOVERNMENT PRINTING OFFICE: 1948

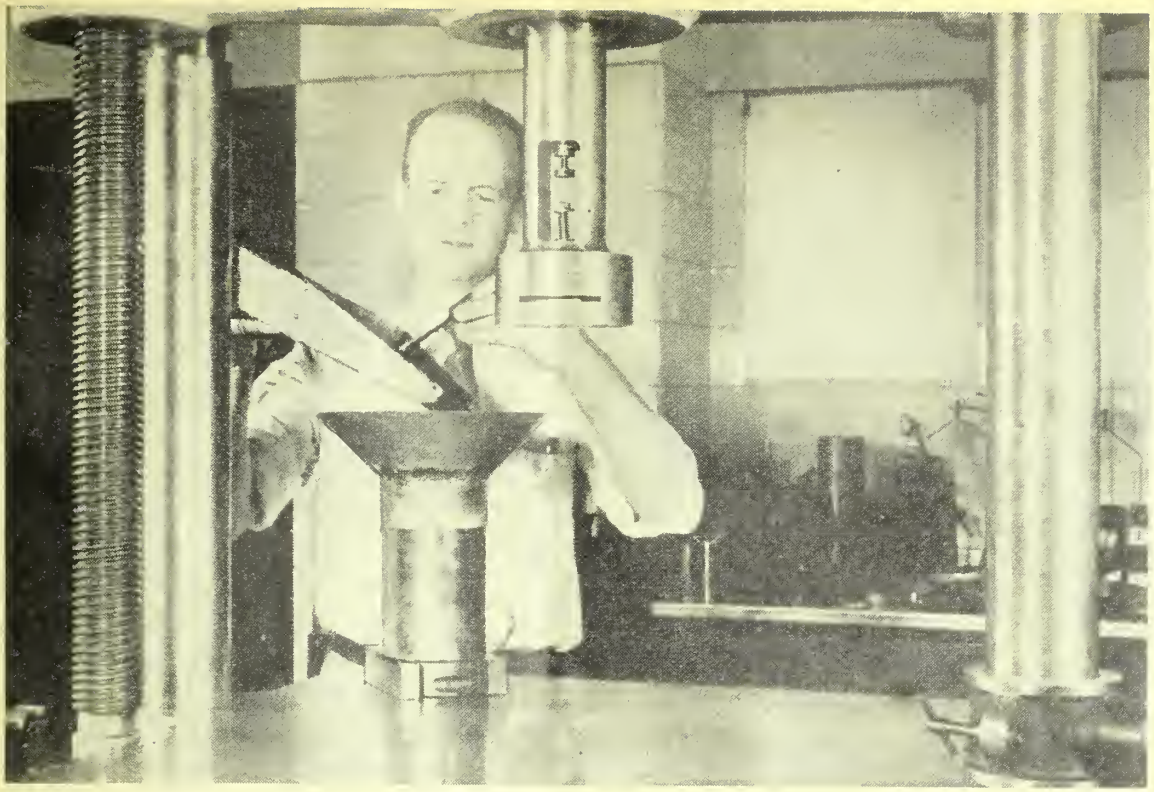

Figure 1.—Filling the 4-inch mold to produce a compressed specimen.

Further Developments and Application of the Immersion-Compression Test

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by J. T. PAULS, Principal Highway Engineer
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Loss of adhesion of bituminous film to the surface of mineral aggregate through the action of moisture, and the resultant stripping, is often a major cause of failure in bituminous pavements. The immersion-compression test was developed to provide a means of determining the resistance of bituminous mixtures to the detrimental action of moisture. This method has definite advantages over the various stripping tests now in use. It provides a quantitative index of the damage caused by moisture, and the test is made on the entire mixture as it might be prepared for use in the road rather than on the single-size fractions of aggregate that are used in stripping tests.

The immersion-compression test involves a comparison of the compressive strengths of molded cylindrical specimens of bituminous mixtures with the strengths of duplicate specimens that have undergone immersion in water for a definite period of time. The test gages the tendency of a mixture to strip by measuring the reduction in strength of the specimens caused by the loss in adhesion of the bituminous film to the aggregate particles. Reports of the initial development of this test have previously been published. This report describes the broader application of the test to mixtures containing a wide variety of bituminous materials, and presents definite procedures and test conditions that have been found appropriate to the various classes of materials.

THIS REPORT covers the recent developments in the procedure for conducting the immersion-compression test and gives additional results of its application to studies of a number of factors affecting the resistance of bituminous paving mixtures to the action of moisture. Earlier work was described in PUBLIC ROADS [1] in 1945. Subsequent studies showed the need for a number of changes in the procedure originally proposed and provided additional data that were presented in a progress report.[2] The essential information in that progress report and the results of more recent studies of mixtures made with cut-back asphalts and slow-curing liquid asphaltic materials are reported herein and make it possible at this time to establish a procedure that appears to be satisfactory.

The immersion-compression test involves a comparison of the compressive strengths of molded cylindrical specimens of bituminous mixtures with those of duplicate specimens which have undergone immersion in water for a definite period of time. All details of the test are controlled as described in the procedure. Essentially the test is intended to

[1] *A test for determining the effect of water on bituminous mixtures*, by J. T. Pauls and H. M. Rex; PUBLIC ROADS, vol. 24, No. 5, July-August-September 1945.

[2] *Application and present status of the immersion-compression test*, by J. T. Pauls and J. F. Goode; Proceedings of the Association of Asphalt Paving Technologists, vol. 16, February 1947.

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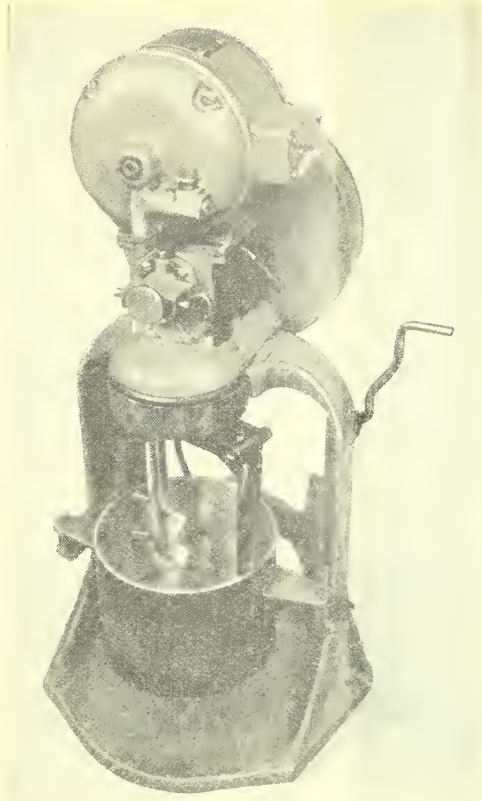

Figure 2.—Mechanical mixer used in preparing individual test batches.

simulate the action of water on a bituminous pavement, and it measures the tendency of a mixture to strip by indicating the loss in strength caused by loss in adhesion of the bituminous film to the aggregate particles. The compressive strength of a cylindrical specimen of a bituminous mixture without lateral support is dependent on the degree of cohesive strength of the mixture, as provided by the bituminous binder; and any damage to the mixture, such as stripping or reduction in the adhesion of the film to the surface of the aggregate particles, results in a measurable loss in the strength of the specimen. For these reasons a compression test of laterally unsupported cylinders is believed to provide a satisfactory means of evaluating the damage caused by water.

This method provides for the testing of bituminous mixtures as used in actual road construction. Thus the water-resistant property of a mixture as a whole is determined and it is also possible to determine the comparative value of each of the constituent parts of the mixture. This is not possible with the more common stripping tests, which are made only on a fraction of the coarse aggregate. It is well known that the sand and filler may have a greater effect on the behavior of a mixture than does the coarse-aggregate fraction. Often the bad effect of a less satisfactory coarse aggregate may be overcome by the use of a good sand or filler or by increasing the density or reducing the size of the pore spaces in the mixtures.

The following details of a procedure for the immersion-compression test as applied to bituminous mixtures are now believed to be satisfactory:

A. Preparation of Specimens

1. Mixing: Thorough mixing in batches just sufficient for one test cylinder.

2. Curing of mixtures before molding:

(a) Hot mixtures: None.

(b) Cold mixtures: 18 hours in oven at 140° F.

3. Test specimens:

(a) Size: 4- by 4-inch cylinders.

(b) Number: Six specimens for each test mixture, three of which are to be tested dry at the same time that the other three are put in water.

4. Molding temperatures:

(a) Hot mixtures: 260° F.

(b) Cold mixtures: 140° F.

5. Molding load (double plunger method) 3,000 pounds per square inch maintained for 2 minutes.

6. Curing after molding: 24 hours in oven at 140° F.

7. Selection of specimens for test groups: Each group of three specimens to have approximately the same average bulk specific gravity.

B. Immersion Conditions

1. Hot mixtures and asphalt emulsion mixtures: 4 days at 120° F. (For a quick test, 1 day at 140° F. may be used.)

2. Cold mixtures (except asphalt emulsion): 4 days at 100° F. (For a quick test, 1 day at 120° F. may be used.)

C. Compression Test Conditions

1. Temperature: 77° F.

2. Rate of deformation: 0.2 inch per minute.

D. Test Results

1. Retained strength of immersed specimens: Taken as the measure of the resistance of the mixture to the action of water, and expressed as a percentage of the compressive strength of the dry specimens.

2. Volumetric swell of immersed specimens: Determined and considered as corollary data to indicate the resistance of the mixture to the action of moisture.

PREPARATION OF SPECIMENS

Mixing.—Most satisfactory results were obtained by mixing in batches of just sufficient size for one test cylinder. In the original work the batches for each of the three specimens of a test group were sampled from a larger batch prepared in a 20-pound capacity laboratory pugmill. Because of segregation, especially in mixtures of the open-graded type, it was very difficult to obtain the desired uniformity among specimens of the same test group. The lack of uniformity that may result is demonstrated by the data given in table 1. These data were obtained from the first and third specimens molded from a 15-pound batch. As indicated by these tests, the first sample is apt to contain more fines than a later sample and therefore result in higher bitumen content and a lower percentage of air voids. Such nonuniformity between individual specimens is usually reflected in the final test results. The differences between the absorptions and between the final compressive strengths were rather wide for the two specimens shown in the table. Because of the difficulty in selecting uniform samples from a larger batch this method has been eliminated from the procedure in the testing of freshly prepared mixtures of the asphaltic concrete type. Instead, the one-batch-per-specimen method has been substituted. Although not necessary for sand-asphalt mixtures, the desirability of its use in the testing of mixtures that tend to segregate is obvious.

The individual batches may be prepared by hand mixing but the use of a small mixing

Table 1.—*Effect of molding from a large batch* [1]

	Specimen [2]	
	First	Third
Bulk specific gravity	2.56	2.52
Air voids—percent	3.8	8.4
Absorption after immersion [3]—percent	0.4	1.1
Compressive strength after immersion—p. s. i.	271	248
Bitumen, CS_2 extraction—percent	5.7	4.5
Aggregate passing—		
½ inch sieve—percent	90	88
⅜ inch sieve—do	76	70
No. 4 sieve—do	54	47
No. 10 sieve—do	33	28

[1] 85–100 penetration asphaltic concrete. Batch for three specimens, 15 pounds.
[2] Each specimen weighed about 4.6 pounds.
[3] 4-day immersion at 77° F.

Table 2.—*Effect of method of curing loose mixtures* [1]

Mixture grade and designation	Curing of loose mixture	Gross loss of original weight of volatile in cut-back—		Compressive strength
		After curing loose mixture	After curing [2] molded specimens	
RC–4 cut-back:		*Percent*	*Percent*	*P. s. i.*
1	None	[3] 14	26	105
2	24 hours at room temperature	32	39	138
3	18 hours in oven at 100° F.	39	46	161
4	4 hours in oven at 140° F.	44	49	162
5	18 hours in oven at 140° F.	57	64	212
MC–3 cut-back:				
6	None	[3] 5	15	46
7	24 hours at room temperature	16	24	68
8	18 hours in oven at 100° F.			81
9	4 hours in oven at 140° F.	30	35	76
10	18 hours in oven at 140° F.	53	56	105

[1] All of same blend. Aggregate retained on No. 10 sieve, trap rock; passing No. 10 sieve, river sand and limestone dust. Bitumen-aggregate ratio, 4.9 percent. Gradation: Passing ¾ inch sieve, 100 percent; passing No. 10 sieve, 21 percent; passing No. 200 sieve, 6 percent.
[2] 24 hours at 140° F.
[3] This loss took place during the mixing operation.

machine is much more desirable. A mixer now in use that has proven satisfactory is shown in figure 2. It is a kitchen-type mixer with a special mixing bowl, a stirrer for throwing the mix to the sides of the bowl, and a scraper for keeping the sides clean. It produces good mixtures with a minimum of crushing. Usually less than 2 minutes of mixing time is sufficient.

Because of differences in types of mixtures and in mixing machines, it is believed desirable to require that mixing be sufficient to obtain a satisfactory coating rather than to specify any particular time.

Curing of mixture before molding.—As stated in the outline of procedure, no curing is required for the hot mixtures (those containing asphalt cement as the binder). For these mixtures, a molding temperature of 260° F., as specified, makes it possible to mold immediately following the completion of mixing.

The cold mixtures containing cut-back asphalts, slow-curing liquid asphalts, and asphaltic emulsions require curing before molding. Both air and oven curing were studied and it was found that oven curing resulted in a loss of volatile material more nearly comparable with the loss that would be expected in good construction practice.

Table 2 gives the loss of solvent occurring in RC–4 and MC–3 cut-back mixtures when cured under a number of conditions.

The data show that curing of the loose mixture for 24 hours at room temperature is not a very effective method for a graded mixture. This curing condition, followed by the oven curing after molding, resulted in a final loss of volatile of 39 percent for the RC–4 mixture and of 24 percent for the MC–3 mixture. These losses were somewhat greater than those that occurred with no curing of the loose mixture, but the amount of curing offered by this method appears to be insufficient because it is believed that good construction procedure for medium-curing cut-back mixtures requires manipulation until the loss of volatile material has exceeded 40 or possibly 50 percent. Curing in an oven at 100° F. for 18 hours and at 140° F. for 4 hours, producing results with the RC–4 mixture that were almost identical, proved to be somewhat more effective in producing the desired amount of loss in volatile material. However, there is some doubt that the 35-percent loss of volatile matter, occurring in the MC–3 mixture under the 4-hour curing at 140° F., is of sufficient magnitude. Increasing the period of curing at 140° F. to 18 hours for this mixture increased the final loss of volatile material to 56 percent, which appears to be adequate.

Curing in an oven at 140° F. for 18 hours was found to be suitable for the RC–4 mixture in that a high amount of volatile material was removed without resulting in a mix too stiff for proper molding.

In tests made on an asphaltic-emulsion mixture it was found that this degree of curing was sufficient to remove all water and still provide a mixture not too difficult to handle. It appears, therefore, that an 18-hour period of curing at 140° F. is satisfactory for all cold mixtures.

Size of test specimens.—Cylindrical test specimens, 4 inches by 4 inches, have proven satisfactory. Specimens 4 inches in diameter by 8 inches in height have been used but these were difficult to handle, and the less stable specimens often failed in removing them from the water bath. Three- by three-inch cylindrical specimens have been used in some instances to conserve materials. Test specimens of this size were found satisfactory for the fine-grained types of mixtures but for the coarse mixtures the test results were not as uniform as for the larger specimens.

Molding temperatures.—Hot mixtures mold satisfactorily at 260° F. This is approximately the temperature of a hot mix after completion of mixing so that little or no delay need occur prior to molding in the case of these materials.

Table 3.—*Effect of vacuum and immersion temperature on test results for asphaltic concretes of same gradation* [1]

Mix designation	¾ inch to No. 10 aggregate [2]	Air voids	Compressive strength, dry	Immersion temperature	After 4-day immersion— Compressive strength		Retained strength ratio		Difference between percentages of retained strength ratios
					No vacuum	Vacuum used	No vacuum	Vacuum used	
Limestone dust mixtures:		*Percent*	*P. s. i.*	*°F.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	
11	Limestone	5.2	222	77	226	218	102	98	4
				100	213	200	96	90	6
				120	215	204	97	92	5
12	Trap	6.2	263	77	247	245	94	93	1
				100	237	216	90	82	8
				120	218	195	83	74	9
13	Granite	6.5	265	77	244	225	92	85	7
				100	215	178	81	67	14
				120	175	154	66	58	8
Silica dust mixtures:									
14	Limestone	5.8	218	77	211	203	97	93	4
				100	190	174	87	80	7
				120	161	129	74	59	15
15	Trap	6.9	251	77	251	241	100	96	4
				100	218	198	87	79	8
				120	136	80	54	32	22
16	Granite	7.2	272	77	241	220	89	81	8
				100	199	164	73	60	13
				120	113	103	42	38	4

[1] Gradation: passing ¾ inch sieve, 100 percent; passing No. 10 sieve, 21 percent; passing No. 200 sieve, 6 percent. Bitumen-aggregate ratio, 4.9 percent. Penetration of asphalt, 85–100.

[2] The sand fraction of the aggregate was river sand in each case.

Figure 3.—Air bubbles on the specimens in the vacuum bath indicate the evacuation of voids.

The cold mixtures—those containing liquid bituminous materials—mold satisfactorily at 140° F., that is, at the temperature as removed from the curing oven. In this group the RC cut-back asphalts and the asphalt emulsions set the temperature for molding. While the MC cut-back asphalts and the SC liquid asphalts can be molded at lower temperature, the RC and emulsion mixtures require this higher temperature.

Molding load (double plunger method).—The double plunger method and a load of 3,000 pounds per square inch held for 2 minutes seem to be a satisfactory procedure for consolidation. Some crushing occurs with soft and fragile aggregate and it may be advisable to use a lower consolidating load for such conditions.

Loading the mold so as to prevent segregation of the aggregate particles is most important. It has been found that segregation may be largely eliminated by the exercise of care in loading and unloading the scoop used in filling the mold. The material should be spread into the loading scoop as uniformly as possible and then scraped into the mold, as illustrated in figure 1, rather than poured. After the mold is filled, the mixture should be leveled off on top and then spaded around the edge with a heated spatula to reduce the tendency for surface voids to form. Rodding is not required for it appears to be unnecessary.

Curing after molding.—After molding, curing at an elevated temperature seems to be desirable for both cold and hot mixtures. In the case of cold mixtures, it brings about more uniformity in the compressive strength by reducing the effect of variations in the degree of curing that occurs during laboratory storage. In the case of hot mixtures, although not losing an appreciable amount of volatile matter, they will increase in strength during storage even at room temperature.

Selection of specimens for test groups.—A small variation in density often results in a large variation in compressive strength. From tests on several specimens from one of the mixtures used in this study, it was found that a 1-percent change in density caused a 10-percent change in compressive strength.

The importance of having the densities approximately the same for the two test groups where their compressive strengths are compared is therefore obvious. The average bulk specific gravity should be obtained for the six specimens for a test, and the three specimens for each group should be selected so that the average specific gravity for each group is, as nearly as possible, equal to the average for all six specimens.

In determining the bulk specific gravity of a specimen three different weights are obtained. The specimen is first weighed in air and then placed in a water bath at room temperature where it is allowed to remain for a few minutes. If the specimen tends to absorb water at a fast rate, as indicated by a rapid formation of air bubbles, it is allowed to soak until the rate of absorption decreases to a small value. The specimen is then removed from the water and surface dried with a damp absorptive cloth, as illustrated on the cover of this issue. The specimen is next weighed in air and then in water. The difference between these two weights in grams is taken as the bulk volume of the specimen in cubic centimeters. Bulk specific gravity is calculated by dividing the initial weight of the unsoaked specimen in grams by the bulk volume of the specimen.

Table 4.—*Effect of vacuum on test results for asphalt [1] mixtures of various gradations*

Mix designation	Kinds of aggregate [2]		Gradation of aggregate			Bitumen-aggregate ratio	Air voids	Compressive strength, dry	After 4-day immersion at 120° F—				Difference between percentages of retained strength ratios
	¾ inch to No. 10 fraction	Sand fraction	Maximum size	Amount passing—					Compressive strength		Retained strength ratio		
				No. 10 sieve	No. 200 sieve				No vacuum	Vacuum used	No vacuum	Vacuum used	
				Per cent	*Per cent*	*Per cent*	*Per cent*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Per cent*	*Per cent*	
11	Limestone	River sand	¾ in.	21	6	4.9	5.2	222	215	204	97	92	5
12	Trap	do	¾ in.	21	6	4.9	6.2	263	218	195	83	74	9
13	Granite	do	¾ in.	21	6	4.9	6.5	265	175	154	66	58	8
17	Granite	do	¾ in.	45	11	6.0	6.0	408	326	216	80	53	27
18	None	do	No. 10	100	15	12.0	8.2	322	228	116	71	36	35
19	None	Limestone	No. 10	100	22	7.0	6.3	518	429	224	83	43	40

[1] 85–100 penetration.

[2] The dust fraction was limestone dust in each case.

IMMERSION CONDITIONS

Considerable laboratory work was done to determine the most suitable immersion condition. Among the more important factors studied were the following:

1. Use of vacuum to bring about quick absorption of water into the specimen.
2. Temperature of the immersion storage bath.
3. Period of immersion.

Figure 4.—The automatic water bath used in the immersion test.

Use of vacuum on hot mixtures found unsatisfactory.—Considerable work was done using vacuum as a preliminary treatment to produce saturation of the cylinders before immersion in the storage bath. The vacuum bath used in this investigation is shown in figure 3. In these tests the temperature of the water in the vacuum bath was 77° F. and the degree of vacuum was 27 inches of mercury. The specimens were kept in this bath until the air ceased to pass off from their surfaces.

The time required for this operation was about 20 minutes. Swell and strength tests were made immediately after saturation on a number of specimens covering several types of hot mixtures, and it was found that there was no swell or loss in strength. It was concluded, therefore, that the use of the vacuum had no initial detrimental effect on the specimens.

The effect of pre-saturation, by use of a vacuum, on the final results of several asphaltic mixtures of the hot type is given in tables 3 and 4. The six asphaltic concrete mixtures of the same gradation shown in table 3 were of the open-graded type that normally tend to absorb water quite readily. These mixtures were subjected to three different immersion temperatures. In all instances the use of a vacuum resulted in reducing the values of retained strength. Greater reductions occurred, in general, with the higher immersion temperatures. The reductions in percentage of retained strength averaged 4.7, 9.3, and 10.5 for immersion temperatures of 77°, 100°, and 120° F., respectively.

Table 4 shows the effect of using a vacuum on mixtures containing aggregate of various gradations. It includes three of the asphaltic concretes of the open-graded type given in table 3, an asphaltic concrete high in percentage of fine aggregate, a sheet asphalt, and an artificial sand mixture. All mixtures contained limestone dust as filler.

The latter three mixtures (Nos. 17, 18, and 19), containing large proportions of fine aggregate, were of types that under normal conditions absorb little or no water. These mixtures were not affected greatly when immersed at 120° F. without pre-saturation, but when subjected to the action of a vacuum prior to the immersion they were severely affected. The use of a vacuum on these mixtures resulted in decreasing the percent-

ages of retained strength by values ranging from 27 to 40. These decreases are much greater than those ranging from 5 to 9 for the open-graded type mixtures Nos. 11, 12, and 13.

If the percentages of retained strength of the six mixtures are compared on the basis of the non-vacuum tests, it will be observed that the three mixtures high in fines (Nos. 17, 18, and 19) are not greatly inferior to the excellent open-graded limestone mixture (No. 11) and that they are somewhat better than the open-graded mixture which contains the granite (No. 13).

On the other hand, comparisons on the basis of tests in which a vacuum was used show that the three mixtures containing high percentages of fine aggregate are definitely inferior to the open-graded mixture containing limestone and somewhat inferior to the one containing granite. The sheet asphalt mixture (No. 18) and the all-limestone artificial sand mixture (No. 19) appear to be unsatisfactory for use as road materials. This does not seem to be in line with expected field behavior because fine-grained mixtures normally have

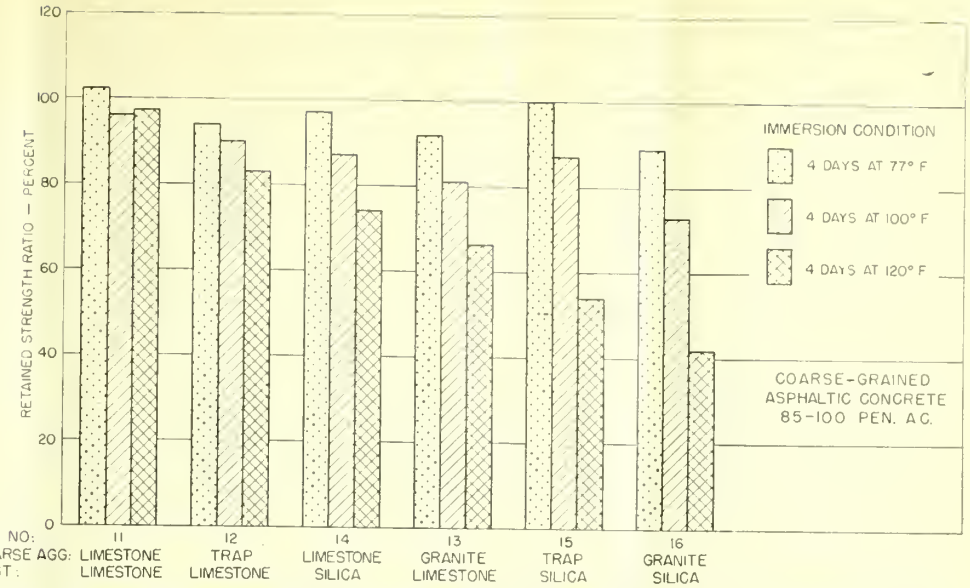

Figure 5.—Effect of immersion temperature on retained strength ratio of coarse-grained asphaltic concrete.

Table 5.—*Comparison of test results for 1-day immersion at 140° F. with those for 4-day immersion at 120° F. for asphaltic concrete mixtures*

Type of asphalt and mix designation	Coarse aggregate in mixture [1]	Gradation of aggregate: [2] Amount passing—		Bitumen-aggregate ratio	Air voids	Compressive strength, dry	Effect of immersion on—					
							Compressive strength		Volumetric swell		Retained strength ratio	
		No. 10 sieve	No. 200 sieve				1 day at 140° F.	4 days at 120° F.	1 day at 140° F.	4 days at 120° F.	1 day at 140° F.	4 days at 120° F.
50-60 penetration:		*Percent*	*Percent*	*Percent*	*Percent*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*
20	Limestone	21	6	4.9	4.1	399	354	332	0.6	0.4	89	83
21	Trap	21	6	4.9	5.6	478	352	354	.8	.9	74	74
22	Granite	45	11	6.0	5.7	679	536	529	.5	.7	79	78
23	do	21	6	4.9	6.5	472	292	298	1.3	1.3	62	63
85-100 penetration:												
11	Limestone	21	6	4.9	5.2	222	211	215	.3	0	95	97
12	Trap	21	6	4.9	6.2	263	216	218	.8	.2	82	83
17	Granite	45	11	6.0	6.0	408	307	326	.7	.8	75	80
13	do	21	6	4.9	6.5	265	187	175	1.1	1.0	71	66
120-150 penetration:												
24	Limestone	21	6	4.9	4.0	187	180	175	.1	0	96	94
25	Trap	21	6	4.9	5.3	234	197	195	.4	.5	84	83
26	Granite	45	11	6.0	5.7	342	280	279	.6	.5	82	82
27	do	21	6	4.9	5.9	224	176	169	.8	1.0	79	75

[1] Retained on No. 10 sieve. Aggregate passing No. 10 sieve was river sand and limestone dust.
[2] Maximum size, ¾ inch.

Table 6.—*Effect of period of immersion on test results for hot mixtures* [1]

Mix designation	¾ inch to No. 10 aggregate [2]	Gradation of aggregate			Bitumen-aggregate ratio	Air voids	Compressive strength, dry	Retained strength ratio after immersion at 120° F. for—			
		Maximum size	Amount passing—					1 day	4 days	14 days	35 days
			No. 10 sieve	No. 200 sieve							
Limestone dust mixtures:			*Percent*	*Percent*	*Percent*	*Percent*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*
11	Limestone	¾ in.	21	6	4.9	5.2	222	95	97	----	----
12	Trap	¾ in.	21	6	4.9	6.2	263	94	83	----	----
17	Granite	¾ in.	45	11	6.0	6.0	408	85	80	71	63
18	----	No. 10	100	15	12.0	8.2	322	----	71	66	59
13	Granite	¾ in.	21	6	4.9	6.5	265	83	66	54	47
Silica dust mixtures:											
14	Limestone	¾ in.	21	2	4.9	5.8	218	89	74	----	----
15	Trap	¾ in.	21	6	4.9	6.9	251	87	54	----	----
16	Granite	¾ in.	21	6	4.9	7.2	272	68	42	----	----

[1] 85-100 penetration asphalt cement.
[2] The sand fraction of the aggregate was river sand in each case.

a high resistance to the infiltration of moisture and for that reason usually show good service behavior.

Hence, because of its failure to reflect expected service behavior, it must be concluded that the vacuum process does not belong in the test procedure.

Conditions of immersion for hot mixtures.—An extended study to determine the most suitable temperature and period of immersion for hot mixtures has been made. Temperatures of 77°, 100°, 120°, and 140° F., and periods of immersion of 1, 4, 14, and 35 days were investigated. Accurate control of temperatures was obtained by using the automatic water bath shown in figure 4.

In the early work 77° F. had been used, but it developed that this low temperature of bath did not show the significant differences between those mixtures that were known to have high resistance to the action of moisture and those known to have low resistance. The more recent study substantiates this conclusion and fully establishes that this temperature fails to differentiate between aggregates having satisfactory and those having unsatisfactory behavior in use. The effect of the various immersion temperatures on coarse-grained asphaltic concretes are shown in table 3 and figure 5.

It will be noted that a very narrow range in test values is obtained with an immersion temperature of 77° F. At 100° F. the results show a somewhat greater differentiation between the several mixtures, but do not provide as positive a one as is desirable. At 120° F., however, the strength retention of the mixtures containing aggregates that are known to be superior is consistently higher than that of the inferior mixtures.

Results of immersion tests at 140° F. for 1 day and at 120° F. for 4 days are compared in table 5. These results show a very close agreement between the test values for the two immersion conditions. It is believed, therefore, that where quick tentative results are

Table 7.—*Effect of immersion temperature on test results for liquid asphaltic mixtures*

Mix designation	Grade of bituminous material	Kinds of aggregate [1]		Gradation of aggregate			Bitumen-aggregate ratio	Air voids	Compressive strength				Volumetric swell after—		Retained strength ratio after—				
		Retained on No. 10 sieve	Dust fraction	Maximum size	Amount passing—				Dry	After 4-day immersion at—			4 days at 77° F.	4 days at 100° F.	4 days at 77° F.	4 days at 100° F.	4 days at 120° F.	1 day at 100° F.	1 day at 120° F.
					No. 10 sieve	No. 200 sieve				77° F.	100° F.	120° F.							
RC cut-back:				*Inch*	*Percent*	*Percent*	*Percent*	*Percent*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
28	RC-4	Limestone	Limestone	¾	21	6	4.9	6.1	166	151	134	124	0.3	0.8	91	81	75	91	86
29	RC-4	Trap	do	¾	21	6	4.9	6.3	215	172	135	108	.3	1.0	80	63	50	80	68
30	RC-4	Granite	do	¾	21	6	4.9	6.9	230	150	94	67	1.2	2.6	65	41	29	56	39
31	RC-4	Trap	Silica	¾	21	6	4.9	6.7	204	162	47	0	.6	5.2	79	23	0	----	----
MC cut-back:																			
32	MC-3	Limestone	Limestone	¾	21	6	4.9	6.6	84	79	78	75	.3	.3	94	93	89	----	----
33	MC-3	Trap	do	¾	21	6	4.9	6.5	104	83	61	45	.4	1.2	80	59	43	----	----
34	MC-3	Granite	do	¾	21	6	4.9	7.1	128	54	37	30	1.8	2.7	42	29	23	46	30
35	MC-3	Trap	Silica	¾	21	6	4.9	6.9	104	53	2	0	1.6	9.2	51	2	0	----	----
36	MC-3	Gravel	do	½	60	10	8.0	6.9	75	59	12	----	1.0	6.6	79	16	----	67	29
SC asphalt (high loss [2]):																			
37	SC-2	do	Limestone	½	60	10	8.0	4.8	32.9	29.6	25.5	----	.8	1.4	90	78	----	----	----
38	SC-3	do	do	½	60	10	8.0	5.0	49.6	38.8	32.6	----	.9	1.6	78	66	----	----	----
39	SC-4	do	do	½	60	10	8.0	4.6	78.9	63.3	44.6	----	.9	1.7	80	57	----	----	----
40	SC-2	do	Silica	½	60	10	8.0	5.6	28.4	27.5	15.7	----	.5	2.5	97	55	----	----	----
41	SC-3	do	do	½	60	10	8.0	6.4	33.3	29.2	20.9	----	.7	2.2	88	63	----	----	----
42	SC-4	do	do	½	60	10	8.0	5.4	66.0	54.9	32.2	----	.7	2.2	83	49	----	----	----
SC asphalt (low loss [3]):																			
43	SC-2	do	Limestone	½	60	10	8.0	4.9	26.1	23.2	22.3	----	.8	1.2	89	85	----	----	- -
44	SC-3	do	do	½	60	10	8.0	4.9	38.8	34.2	27.3	----	.7	1.4	88	70	----	----	----
45	SC-4	do	do	½	60	10	8.0	4.7	71.4	57.9	41.6	----	.7	1.4	81	58	----	----	----
46	SC-2	do	Silica	½	60	10	8.0	6.0	21.1	18.9	11.3	----	.6	2.8	90	54	----	----	----
47	SC-3	do	do	½	60	10	8.0	6.7	26.9	24.1	13.9	----	.8	3.2	90	52	----	----	----
48	SC-4	do	do	½	60	10	8.0	5.2	60.7	48.7	24.7	----	.7	2.8	80	41	----	----	----

[1] The sand fraction of the aggregate was river sand in each case.
[2] Prepared in laboratory by blending a 279-penetration asphalt with a high-loss gas oil.
[3] Prepared in laboratory by blending a 279-penetration asphalt with a low-loss gas oil.

Table 8.—*Results of alternate drying and wetting tests on MC-2 mixtures* (*see text, page 116*)

Coarse aggregate,[1] and mix designation	Additive in cut-back	Compressive strength						Volumetric swell					Final moisture content					Retained strength [2] ratio	Compressive strength ratio [3]			
		Dry	4-day immersion at 77° F.	4-day immersion at 77° F. followed by alternate drying and wetting [4]			10-month exposure to weather [5]	4-day immersion at 77° F.	4-day immersion at 77° F. followed by alternate drying and wetting			10-month exposure to weather	4-day immersion at 77° F.	4-day immersion at 77° F. followed by alternate drying and wetting			10-month exposure to weather		4-day immersion at 77° F. followed by alternate drying and wetting			10-month exposure to weather
				5 cycles, 74 days	10 cycles, 144 days	20 cycles, 284 days			5 cycles, 74 days	10 cycles, 144 days	20 cycles, 284 days			5 cycles, 74 days	10 cycles, 144 days	20 cycles, 284 days			5 cycles, 74 days	10 cycles, 144 days	20 cycles, 284 days	
Pit gravel mix:		*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
64	None	43	33	53	68	88	76	0.4	0.7	1.1	1.0	0.5	1.6	2.1	2.5	2.5	1.9	77	123	158	205	177
65	*A*	45	36	52	66	80	75	0	.3	.4	.4	.3	1.4	1.8	2.1	2.5	1.9	80	116	147	178	167
66	*D*	38	36	53	70	84	81	.3	1.2	1.4	1.4	.4	1.5	2.3	2.3	2.4	2.1	95	139	184	221	213
67	*B*	39	39	76	93	116	86	0	.1	.2	.2	.1	1.2	1.8	2.2	2.1	1.5	100	195	238	297	221
Granite mix:																						
68	None	64	35	41	44	67	60	1.3	2.9	3.0	2.6	1.2	1.5	2.6	2.7	2.4	2.1	55	64	69	105	94
69	*A*	61	50	67	74	93	83	.1	.3	.4	.4	.3	1.0	1.4	1.7	1.9	1.5	82	110	121	152	136
70	*D*	61	57	80	93	118	92	.5	1.3	1.2	1.1	.6	1.2	1.7	1.8	1.8	1.6	93	131	152	193	151
71	*B*	61	59	101	112	141	100	.1	.3	.4	.5	.4	1.0	1.4	1.5	1.6	1.3	97	166	184	231	164

[1] The sand fraction was river sand and the dust fraction silica in each case. Gradation of the aggregate was as follows: Passing ½ inch sieve, 100 percent; passing No. 10 sieve, 40 percent; passing No. 200 sieve, 10 percent. Bitumen-aggregate ratios: 6.0 percent for the pit gravel; 5.7 percent for the granite mixture.
[2] 4-day immersion at 77° F.
[3] Ratio of final strength to the original strength dry.
[4] A cycle of alternate drying and wetting consisted of 10 days' drying in laboratory air and 4 days' immersion in water at 77° F.
[5] The test cylinders were buried outdoors in a sand bed with top surfaces exposed. Before testing, they were immersed in water at 77° F. for 4 days.

desired, the results of tests made after 1-day immersion at 140° F. may be used with confidence.

Table 6 presents data on the effect of various periods of immersion at a temperature of 120° F. The 1-day results at 120° F. were approximately the same as the 4-day results at 100° F. (given in table 3 except for mixes Nos. 17 and 18) and failed to provide a sufficiently sharp evaluation. On three of the mixtures, the period of immersion was extended to 14 and 35 days in order to determine the reliability of results obtained in the shorter and more practical periods. The results of these tests indicate that, while some additional loss occurs, the immersions for 14 and 35 days show the same order of resistance by the various mixtures to water action as immersion for 4 days.

Conditions of immersion for cold mixtures.—A study was made to determine suitable immersion conditions for the liquid asphaltic mixtures. In table 7 are shown the data obtained on RC–4 and MC–3 mixtures with an immersion period of 4 days at temperatures of 77°, 100°, and 120° F. Also included are percentages of retained strength for a 1-day immersion period at temperatures of 100° and 120° F. The 4-day immersion results are shown in figure 6. If the results for mixtures Nos. 31 and 35 were not considered, it would appear that both temperatures 77° and 100° F. produce test values, for a 4-day immersion period, that permit satisfactory differentiation between the several mixtures. Mixtures Nos. 31 and 35 contained silica dust as a filler and the 4-day results at 100° F., for these two mixtures, are believed to be more nearly in agreement with expected field behavior than those in which the immersion temperature was 77° F. The results for a 4-day immersion period at 120° F. indicate that this condition is too severe, especially for mixtures containing silica dust. Specimens from mixtures Nos. 31 and 35 disintegrated in less than 4 days at this high temperature.

Several of the cut-back mixtures were tested after being immersed for a 1-day period at temperatures of 100° and 120° F. The results for the 1-day test at 100° F. compare very favorably with those for the 4-day test at 77° F. Also, there is very little difference between the results for a 1-day period at 120° F. and a 4-day period at 100° F. It therefore appears that when there is need for early test results, a 1-day immersion at 120° F. may be used for the cut-back mixtures.

Table 7 also gives data for mixtures containing SC–2, 3, and 4 materials. Both the slow and the more rapid curing SC materials were used in this study. Analyses of these materials are given in table 9. Again, as occurred with the cut-back mixtures, the use of silica dust as a filler resulted in mixtures which were affected to a much greater degree by the change in immersion temperature from 77° to 100° F. than were the mixtures in which limestone dust was used. Also, as with the cut-back mixtures, the results after 4-day immersion at 100° F. appear to conform more nearly to expected service behavior than do the results at the lower temperature.

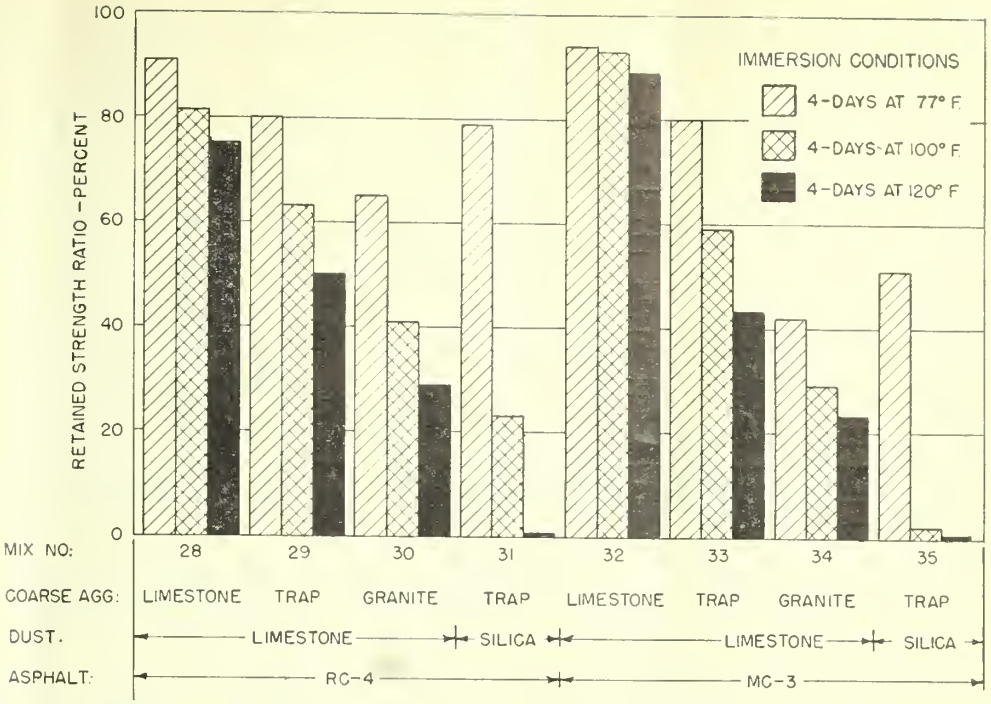

Figure 6.—Effect of immersion temperature on retained strength ratio of cut-back asphaltic mixtures.

Because of an insufficient quantity of these SC materials, specimens were not prepared for the 1-day immersion period. However, in view of the good correlations obtained for the other bituminous materials, it can be expected that the results after a 1-day immersion period at 120° F. would agree very closely with those after a 4-day immersion period at 100° F. and that it would be permissible to substitute this shorter period of test when necessary.

A limited amount of work with asphaltic-emulsion mixtures was included in this study to determine if any changes in procedure were necessary for handling this type of cold mixture. It was found that the same curing procedure used for the cut-back and slow-curing bituminous materials was also satisfactory for the emulsion mixtures. However, since the 18 hours of curing at 140° F. removed all the moisture from the mixtures, leaving a penetration grade of asphalt, it appeared desirable to make the immersion condition for this type of cold mix the same as for hot mixes; that is, 4 days at 120° F.

COMPRESSIVE TEST CONDITIONS

In bringing the dry specimens to the test temperature of 77° F., it was found that 2 hours in an air bath at this temperature was sufficient when the specimens, after curing, were permitted to cool overnight at room temperature. Specimens placed in the air bath directly from the oven require about 4 hours to bring them to the test temperature. Because of the higher conductivity of water, the immersed specimens required only 1 hour in water at 77° F. to bring them to the proper temperature for testing.

It is believed that the rate of deformation should vary with the height of the test specimen. A rate of 0.05 inch per minute per inch of height has been found to be suitable. This gives a rate of 0.2 inch per minute for the standard 4-inch specimen. A few tests have been made on 3- by 3-inch cylinders, and in these tests the rate was 0.15 inch per minute.

SOME APPLICATIONS OF THE TEST

The following summary describes some of the more important results that have been obtained with the immersion-compression test, and the nature of the problems that may be studied to advantage by this method of test.

From the work done so far, information has been obtained on a number of factors that affect the behavior of bituminous mixtures. In certain instances the results are conclusive while for some factors more work needs to be done.

Table 9.—*Analyses of slow-curing liquid asphaltic materials*

	Grade					
	SC–2	SC–2	SC–3	SC–3	SC–4	SC–4
Relative volatile content	High	Low	High	Low	High	Low
Furol viscosity at 140° F.—seconds	170	145	373	412		
Furol viscosity at 180° F.—do					178	205
Flash point—° F.	255	360	270	365	280	380
Loss at 325° F., 50 gm., 5 hrs.—percent	8.8	2.6	7.1	2.1	5.3	1.9
Float test on residue at 122° F.—seconds	38	30	58	46	84	71
Asphalt content of 100 penetration—percent	71	67	77	75	81	80
Ductility of residue at 77° F.—cm	165	165	195	200+	185	200+

Table 10.—*Results of a special filler investigation of 85–100 penetration asphaltic concretes*

Mix designation	Aggregate passing No. 200 sieve [1]	Bitumen-aggregate ratio	Air voids	Compressive strength, dry	After 4-day immersion at 120° F.	
					Compressive strength	Retained strength ratio
		Percent	*Percent*	*P. s. i.*	*P. s. i.*	*Percent*
50	Fly ash	6.5	3.6	212	212	100
51	do	5.5	6.5	224	226	101
52	Limestone dust	6.5	5.5	259	242	93
53	do	5.5	8.2	256	225	88
54	Trap-rock dust	6.5	5.7	264	246	93
55	do	5.5	8.5	245	227	93
56	Silica dust No. 2	6.5	3.9	251	213	85
57	do	5.5	6.9	281	189	67
58	Silicia dust [2]	6.5	4.5	252	198	79
59	do	5.5	8.4	261	144	55

[1] Aggregate retained on No. 200 sieve, trap rock and siliceous sand. Gradation: Passing ½ inch sieve, 100 percent; passing No. 10 sieve, 37 percent; passing No. 200 sieve, 7 percent.
[2] Same silica dust used as in the mixtures of tables 3 and 6.

Table 11.—*Effect of additives when used with rapid-curing cut-back bituminous mixtures* [1]

Mix designation	Grade of cut-back material	Additive used with—		Air voids	Compressive strength, dry	After 4-day immersion at 120° F.[2]			
		Bituminous material	Aggregate			Compressive strength, wet	Water absorbed	Volumetric swell	Retained strength ratio
				Percent	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*
60	RC-2	None	None	11.5	184	19	9.6	10.2	10
61	do	None	*T*	10.2	201	159	4.3	1.1	79
62	do	*B*	None	11.4	176	106	5.9	2.7	60
63	RC-3	*F*	None	11.9	238	107	6.9	3.7	45

[1] Chert gravel mixtures using a bitumen-aggregate ratio of 9.3 percent.
[2] Normally a temperature of 100° F. is used for evaluating cut-back mixtures.

Results demonstrate the importance of the aggregates on the behavior of the bituminous mixtures. There is a wide range in the film-retention properties of the aggregates commonly used in bituminous road construction. This is not only true as between the general types but there is a wide range within a particular aggregate type. Limestone, in general, has high resistance to the stripping action of water, yet there may be a considerable range in quality between limestones from different sources because of impurities such as silica, clay, or iron. Similar differences in the quality of trap rock have been found, although, in general, this type of stone possesses high film-retention properties. Granite and quartzite are other types of stone that have been tested. As would be expected, granites range from good to bad and quartzite was found to be the least satisfactory of aggregates studied.

Absorption is a property of the aggregate causing increased adhesion of the bituminous film. Stone having high absorption, such as caliche (as found in New Mexico), soft limestone, and limerock, although they are hard to coat, retain the bituminous film very well.

The quality and quantity of the sand and filler used are shown to have considerable effect on the resistance to the action of moisture. In many cases these materials may be more important than the coarser fractions. A high percentage of fines of good quality may often be used to overcome the adverse effect of a less satisfactory coarse aggregate.

Test results given in table 3 show the comparative effect of limestone dust and silica dust when used in asphaltic concrete mixtures. For the three conditions, covering coarse aggregates of limestone, trap, and granite, the substitution of silica dust for limestone dust as a filler resulted in reducing the percentage of retained strength (4-day immersion at 120° F.) by 23 or more. The silica dust used in these tests was analyzed and found to be 99.7 percent pure SiO_2. It is possible that this silica dust would have been more satisfactory if it had contained more impurities.

In table 10 are shown the results of tests on a series of bituminous concrete mixtures in which the variables were the type and the quantity of filler material. The superiority of the fly-ash, limestone, and trap-rock dust fillers over both of the two silica-dust fillers is clearly shown. Increasing the bitume content is shown to be beneficial for th mixtures composed of the less satisfactor materials but to have little effect on th mixtures composed of the better materials.

The effectiveness of certain chemical addi tives on the adhesion of bituminous films t aggregates has been studied by means of thi test. In this work, the test has proven effec tive in showing the comparative value o these additives and the amounts required.

Table 11 gives the results with and without the use of chemical additives on bituminous mixtures containing a chert gravel of unsatisfactory quality. An increase in retained strength together with a great reduction in swell is shown to result from use of these additives.

In table 8 (on page 114) are shown the results of alternate drying and wetting tests on two groups of bituminous mixtures containing treated and untreated MC–2 cut-back asphalts. Three anti-stripping additives were used as treatments, in concentrations of 2 percent for additive *A* and 1 percent for additives *B* and *D*. The coarse aggregate consisted of a pit gravel for the first group of mixtures and a granite for the second group. The table gives data for various periods up to 284 days during which the specimens, after having been initially immersed for 4 days, were subjected to repeated cycles of alternate drying and wetting. Each cycle consisted of a 10-day period of drying in laboratory air followed by a 4-day period of immersion in water at 77° F. Also included in table 8 are the results of tests made on specimens that had been buried outdoors with top surfaces exposed to the weather for a 10-month period. In addition to the above tests, compressive strength data were obtained for various periods of dry storage in laboratory air up to 284 days. These data are given in table 12. The compressive strength results for all conditions of storage and water exposure are summarized in the form of bar diagrams in figure 7.

A study of the data shows that additive *A*, when used in the cut-back for the gravel mixture, provided no improvement over the untreated cut-back. However, when used in the granite mixture this additive resulted in compressive strengths which, for all conditions of wet storage, were considerably higher than the corresponding strengths for the mixture containing no additive. Additive *D* did pro-

Table 12.—*Effect of length of storage in laboratory air on compressive strength of MC–2 mixtures*

Coarse aggregate,[1] and mix designation	Additive	Compressive strength after storage in air at room temperature				Compressive strength ratio [2] after storage in air at room temperature			
		0 days	74 days	144 days	284 days	0 days	74 days	144 days	284 days
Pit gravel mix:		*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*
64	None	43	82	93	121	100	191	216	281
65	*A*	45	78	92	126	100	173	204	280
66	*D*	38	82	92	116	100	216	242	305
67	*B*	39	80	91	115	100	205	233	295
Granite mix:									
68	None	64	98	115	140	100	153	180	219
69	*A*	61	91	105	129	100	149	172	211
70	*D*	61	95	114	134	100	156	187	220
71	*B*	61	95	111	134	100	156	182	220

[1] The sand fraction was river sand and the dust fraction was silica in each case.
[2] Ratio of final strength to original strength.

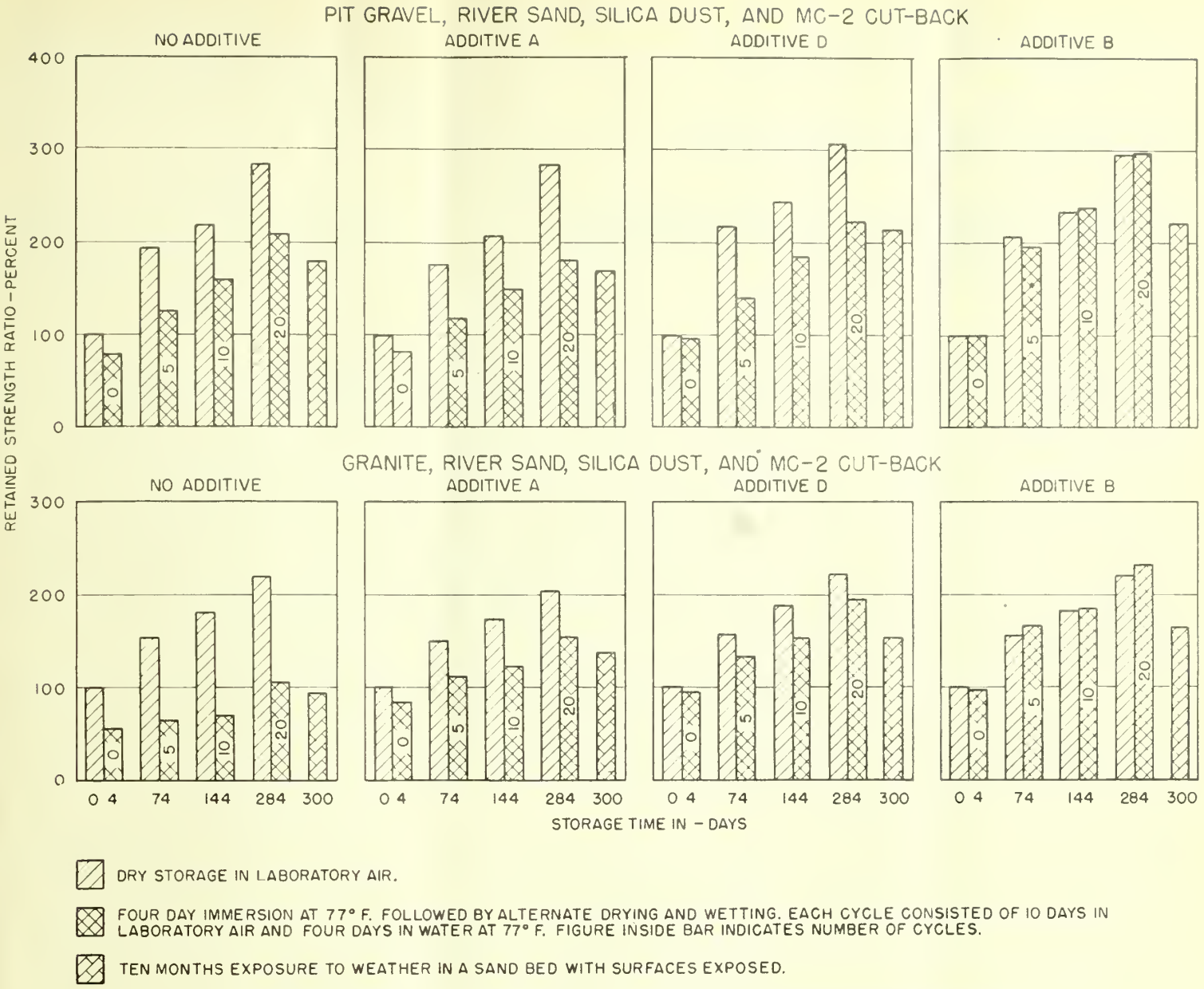

Figure 7.—Effect of additives on retained strength ratio in exposure tests over long periods of time.

vide improvement in the gravel mixture and was somewhat more effective than additive *A* in the granite mixture. Additive *B* was very effective for both mixtures. In the mixtures where this additive was used, the compressive strengths after the various periods of alternate drying and wetting were almost equal to or were greater than the compressive strengths for corresponding periods of dry storage. Considering the extended period over which these tests were made, the beneficial effect of certain additives, when used with medium-curing cutback asphalt, would appear to be fairly permanent.

Table 13.—*Results of tests on mixtures*[1] *containing asphalts from different sources*

Asphalts			Immersion-compression test					
Source	Penetration	Reaction to Oliensis test	Mix designation	Air voids	Compressive strength, dry	After 4-day immersion at 120° F.		
						Compressive strength	Retained strength ratio	Volumetric swell
				Percent	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*
A	89	Positive	72	7.0	325	246	76	0.5
B	93	Negative	13	6.5	265	175	66	1.0
C	78	do	73	6.5	287	153	53	2.0
D	85	do	74	6.5	297	148	50	2.1

[1] Mixtures contained granite, river sand, and limestone dust. Gradation: Passing ¾-inch sieve, 100 percent; passing No. 10 sieve, 21 percent; passing No. 200 sieve, 6 percent. Bitumen-aggregate ratio, 4.9 percent.

In connection with figure 7, it is of interest to note that during dry storage the pit-gravel mixtures increased in strength at a much greater rate than the granite mixtures. The pit gravel had an absorption of 1.7 percent as compared with an absorption of only 0.6 percent for the granite. Since absorption of bituminous material is gradual and continues over a long period of time, it is believed that the difference in absorption between the two aggregates is largely responsible for the difference in the rate at which their mixtures increase in strength.

The use of inferior aggregates is not necessarily the cause of all stripping failures. Many of the failures are probably caused by asphalts having inferior adhesive qualities. This is indicated by the data in table 13. The results given in this table show that asphalts of the same grade from different sources may vary widely in their resistance to the action of moisture.

A Rapid Method of Testing Materials for Alkali-Aggregate Reaction

BY THE DIVISION OF PHYSICAL RESEARCH PUBLIC ROADS ADMINISTRATION

Reported by

D. O. WOOLF, Senior Materials Engineer and T. R. SMITH, Senior Scientific Aid

The generally accepted test method for determining the susceptibility of aggregates to reaction with the alkali in cement requires many months of observation and involves a number of difficulties. Accelerated tests heretofore proposed are found lacking in both simplicity and reliability.

The authors have developed a simple test method considered suitable for detection of reactive materials in a period of 1 month or less. Mortar or concrete, made with the aggregate under test and a high-alkali cement, is sealed in a pint mason jar, which is stored in an inverted position. The jar is examined daily until multiple cracking of the glass occurs, or until a reasonable age is reached without cracking. The method is suitable for field use since the test is simple and of relatively short duration, and the equipment is inexpensive and readily obtained.

THE GENERALLY accepted method of testing aggregate for reaction with the alkali in portland cement follows suggestions made by T. E. Stanton in 1941.[1] In this method, the aggregate is crushed, if necessary, to sand size, and used with high- or low-alkali cement to prepare a 1:2 mortar. The mortar is cast into bars about 11¼ inches long with a 1- or 2-inch square cross section. These bars are cast with gage plugs positioned in the ends to give a gage length of 10 inches in the same manner as in the cement bars for the standard method of test for autoclave expansion of portland cement, A. S. T. M. Method C 151.[2] The bars are measured for length, using a comparator reading to 0.001 inch or less, and stored in moist air. At periodic intervals, the bars are again measured to determine the amount of expansion, if any. With extremely reactive materials, a decision regarding the use of the material with a given type of cement may be made in a few months, but usually the tests must be continued for one or more years.

This method of test has several disadvantages. Possibly the most serious is the time usually required to test a material other than one which is extremely reactive with the alkali. When this time exceeds a month, as it usually does, the method becomes unsuitable for the routine or acceptance testing of aggregates. When samples of coarse aggregate are prepared for test by crushing the material to sand size, all of the crushed material customarily is used in preparing the mortar. In early studies of alkali-reactive aggregates, it was found that the size of the reactive aggregate had a marked effect on the amount of expansion. If the reactive aggregate was crushed fine enough, the mortar would develop little if any expansion. This same behavior may be found in tests of crushed aggregate when finely crushed fragments are included in the sample tested. Other disadvantages of the method include the fragility of the bars, especially when the smaller bar is used, and the difficulty of maintaining a saturated atmosphere for storing the bars. A method frequently used for storing the bars consists of placing the bars above a pool of water in a metal container, and sealing the cover. With a long storage period between consecutive inspections of the bars, and the use of galvanized containers, the loss of water and subsequent drying of the specimens because of the development of pin holes in the container has frequently been reported.

In an attempt to accelerate the reaction between the alkali in the cement and the aggregate, the addition to the mortar of 1 percent of sodium hydroxide, by weight of the cement, has been tried. In some instances mortar so treated has been found to have as much expansion at an age of 3 months as plain mortar developed in a year. Unfortunately, for some reason which has not been determined, a satisfactory correlation between the two methods has not been established. Too frequently, a treated mortar prepared with a high-alkali cement will show an excessive amount of expansion at an early age but plain mortar prepared with the same cement will develop only a small amount of expansion at considerably greater ages. In contrast to this, repeated tests have shown that when a low-alkali cement is used with a reactive aggregate, the addition to the mortar of 1 percent of sodium hydroxide, by weight of the cement, does not change materially the amount of expansion developed by the mortar. In spite of the lack of agreement between the results obtained with the long-time and accelerated tests, the use of mortar treated with sodium hydroxide has been found of value in furnishing at an age of 3 months information as to whether the material under test is unlikely to develop excessive expansion at some greater age when tested in plain mortar. If the treated mortar prepared with high-alkali cement fails to show excessive expansion at an age of 3 months, the aggregate used may be considered to be relatively nonreactive. However, if the mortar shows high expansion in the accelerated test, no definite conclusion can be drawn.

In further search for an accelerated test for the alkali-aggregate reaction, study was given to a method proposed by Alderman, Gaskin, and Vivian.[3] This method involves the warping of a slab of mortar due to differential expansion between the two mortars with which the specimen is prepared. Both mortars are prepared with the same cement. In one mortar, a nonreactive sand is used, and the aggregate under test is used in the second mortar. One mortar is used to form a layer 2- by 4- by ⅛-inch thick, and a layer of equal thickness of the second mortar is placed on and bonded to the first layer as rapidly as possible. When the specimen is removed from the mold at an age of 24 hours, it is placed in moist storage. If the mortar in one layer expands more than the other due to the chemical reaction, the slab will warp. The warping is stated to be preceded by the appearance of damp-looking spots on the surface of the reactive mortar, but the development of the spots is not always followed by warping. In the Australian tests, the specimens were stored at room temperature (58° to 59° F.) and at 110° F. Those prepared with cements containing 0.41 to 1.09 percent alkali, expressed as sodium oxide, and a very reactive aggregate, and stored at the higher temperature, developed prominent spots in 2 to 8 days, and prominent warping in 3 to 37 days. Bar specimens made with these same cements and stored at room temperature showed from 0.33 to 0.68 percent expansion at an age of 12 months.

A representative number of samples of both cements and aggregates were tested with the Australian slab method by the Public Roads Administration. Some of the aggregates for

[1] *Expansion of concrete through reaction between cement and aggregate (Discussion)*, by T. E. Stanton; Proceedings of the American Society of Civil Engineers, vol. 67, p. 1402, September 1941.

[2] *Standard method of test for autoclave expansion of portland cement*, 1946 Book of A. S. T. M. Standards, part II, p. 14.

[3] *A qualitative test for cement-aggregate reaction*, by A. R. Alderman, A. J. Gaskin, and H. E. Vivian; Journal of the Council for Scientific and Industrial Research, Commonwealth of Australia, vol. 18, No. 4, p. 433, Nov. 1945.

these tests were crushed to sand size and all were sieved to pass a No. 30 sieve and be retained on a No. 50 sieve. Ottawa sand graded to meet the requirements given in A. S. T. M. Method C 109 [4] was used as the inert aggregate. This material is graded between the No. 30 and No. 100 sieves. At first, a mold for a single 2- by 4-inch slab was used, as recommended by the originators of the method. The time required to prepare and clean these single molds was considered excessive, and new molds to prepare a slab 2 by 12 inches were made. After the mold had been filled, the specimen was cut into three pieces of equal length. Storage of the test specimens presented a problem, which was finally solved by the use of pint mason jars. The specimen was supported above a small pool of water in the jar by a block of brass or mortar, and the jars stored at 70° to 80° F., or at 120° F.

AUSTRALIAN METHOD TESTED

In the original report the authors state that the "bi-aggregate slabs have not failed to indicate expansive reaction in mortars which at atmospheric temperatures have developed significant expansions in twelve months," and that "in no instances have bi-aggregate slabs given indications of expansive reaction which were not fully substantiated by other observations." The mention of "atmospheric temperatures" is understood to refer to the room temperatures used in the Australian tests. The tests made by the Public Roads Administration do not support these statements fully. Many instances have been found where reactive mortars, as determined by other tests, have failed to develop the spotting or warping which are stated to be indicative in the bi-aggregate slab test. In other cases, very severe spotting has developed on slabs prepared with a mortar which, to an age of 3 years, has shown only an immaterial amount of expansion in bar tests. These slabs did not warp, and the question arises whether the spotting itself may be considered to indicate the alkali-aggregate reaction. When the indication of the reaction was limited to marked warping of the slab, the test was found to be no more rapid than the accelerated bar test in which the mortar was treated with sodium hydroxide. Of these two tests, the latter was preferred due to its quantitative determinations. Attempts to obtain a quantitative measure of warping in the slab test through the use of feeler gages did not prove satisfactory, as the determinations could not be checked with sufficient accuracy.

An attempt to use bi-aggregate slabs 12 inches long has been made with the thought that warping of the slab could be detected at a much earlier age. Some difficulties in the preparation of a slab of uniform density throughout its length have been found, and the handling and storage of slabs 12 inches long and only one-fourth inch thick have presented some problems. No decision as to the suitability of these specimens has been reached.

[4] *Standard method of test for compressive strength of hydraulic cement mortars*, 1947 Supplement to Book of A. S. T. M. Standards. Part II, p. 43.

SIMPLE TEST SOUGHT

The use of large numbers of aggregates obtained from noncommercial sources in the western part of the country caused some thought to be given to the development of a method of test for alkali-aggregate reaction which could be made with a minimum of testing equipment and with no requirements for delicate or expensive apparatus. The development of such a test would permit the examination of samples of aggregate in field offices and reduce the number of samples submitted to the laboratory for more thorough tests. In an effort to develop this test some study was given to the preparation of a hard and brittle container of mortar to enclose the mortar prepared with the aggregate under test. This was on the assumption that, should the mortar under test develop expansion due to chemical reaction, the expansion would crack the external mortar shell and reveal the reactive character of the aggregate at an early age. Difficulties were found in the preparation of the hard mortar containers. Although a hard mortar could be obtained with a 1:2 mix and a water-cement ratio of 0.25, the mortar had to be tamped in place and specimens of uniform density were seldom obtained.

MASON JARS AS CONTAINERS

To furnish a hard and brittle container for samples of mortar under test for alkali activity, the use of mason jars was suggested and tried initially by the junior author. These proved quite successful and a large program of tests to determine the suitability of the method was begun. The mortars included in the first series of tests were prepared with known reactive and nonreactive aggregates, and both high- and low-alkali cements. The jars used had various shapes of horizontal cross section. In some the cross section was circular; in others the cross section was square with rounded corners of short radius; and in a third type the radii of the rounded corners were so large that the cross section approached the circular. Some jars were filled to the neck; others were filled about half full. After the mortar had been placed in the jar, the jar cap was fastened and the jar set aside until the mortar hardened. A small amount of water, about 25 ml., was then added to each jar, and the cover replaced. Later in the study of the method, it was suggested that the jars be inverted after the water had been added to the jar, to insure that the alkali in the mortar would not be leached out. This practice was followed, not necessarily to prevent leaching of the alkali, but to permit more ready detection of cracking in the jar. Another suggestion, based on the behavior of specimens containing reactive aggregates and treated with acid solutions, involved the addition of about 5 ml. of concentrated hydrochloric acid to each jar of mortar. This was added to the mortar immediately after preparation of the test sample. No benefit to the test procedure or acceleration of the reaction was found with this treatment.

Some disappointing results were obtained in the first series of tests. A large number of jars containing mortar prepared with high-alkali cement and a nonreactive aggregate failed at an early age. Other jars containing reactive mortar did not fail within a reasonable length of time. Some jars cracked, while others containing the same mortar did not. The failure of the jars containing a high-alkali cement and an inert sand was traced to a reaction between the cement and the glass of the jar. This was subsequently prevented by coating the jar with an alkali-proof paint. A number of coating materials were tried but best results were obtained with pigmented materials having a chlorinated rubber or a vinyl resin base. Of these two, that with the chlorinated rubber base is preferred. The coating material with the vinyl resin base seems to be tougher and less liable to damage when the jar is being filled, but that with the rubber base has a less objectionable odor, is easy to handle, and dries very rapidly. Several different colors of the rubber base finish were tried, including white, black, and gray. The white coating is not desirable as it interferes with the detection of the dried gel—the end product of the alkali-aggregate reaction—when the mortar is removed for inspection from the jar after failure.

The mechanical strength of the jars undoubtedly has a considerable influence on the results of this test, and the variable results obtained in the first series of specimens possibly were due to the use of different types of jars of variable strength. From observation of the test results obtained, the least desirable type of jar appeared to be that with a square cross section and rounded corners of short radius. Many jars of this type failed only at the junction between the sides and the bottom, and it appears that this is the weakest point in the jar. Jars with a truly round cross section appeared to give the most uniform results but difficulty was found in obtaining these. Those most readily available seem to be the square jar with short radius corners and the jar with heavily rounded corners and a cross section approaching the circular.

OTHER JAR TYPES TESTED

Some jars used in these tests contained striations or mold marks in the glass, and an attempt was made to obtain a jar of more uniform quality. Consideration was given to glass jars used for packaging foodstuffs, such as mayonnaise, and these jars were compared for uniformity with the mason jars of nearly circular cross section. The results obtained with the two types of jar differed so little that it was decided to adopt the latter for use in the test, or the round mason jar when it could be obtained. These investigations indicated that it would be desirable to make more than one jar specimen for each mortar, and the practice of preparing three jars of a kind was adopted. The use of smaller containers such as shell vials was tried and found satisfactory in tests of mortars, but the pint mason jar is preferred as this can be used for testing concrete with a maximum size of aggregate of at least 1 inch.

Storage of the jars at room temperature has been the general practice in most tests conducted to date as this condition probably would be followed in tests in the field. Some tests were made at 120° F. to determine the effect of the higher temperature. Included in this group were test specimens for a number of gravels composed essentially of quartz and chert. Many of these jars broke at a very early age, in several cases at only 2 or 3 days, and it was suspected that the cracking of the jar might have been caused either by increase in volume of the aggregate due to absorption of water, or to expansion of the mortar or concrete due to heat. To investigate these possibilities a series of tests was made on samples of concrete prepared with high-alkali cement, quartz sand, and a gravel containing chert or a crushed granite. Both coarse aggregates were graded between the 1-inch and No. 4 sieves. A batch of concrete, sufficient for six jars, of 1:2:3 mix was prepared, with each coarse aggregate in an oven-dry or saturated condition. Three of each group of six jars were stored at laboratory temperature until the concrete hardened. The other three were placed in a cabinet held at 120° F., as soon as they were prepared. At an age of 24 hours, water was added to all jars, and those which had been stored at laboratory temperature were placed in the warm storage cabinet. The jars were inspected daily for the development of cracking, and the test continued to an age of 71 days.

CAUSES OF FAILURE

The results obtained in this series of tests indicate that failure of the jars may be caused entirely by volume changes due to absorption of water by certain coarse aggregates, or by the thermal volume change of hardened concrete. Consequently, coarse aggregates used in this test should be saturated prior to mixing if they contain types of aggregate which have a large amount of absorption. Included in these types of aggregates are chert and some varieties of limestone. If the jars are stored at an elevated temperature, they should be placed in this storage before the contents of the jar have hardened. It should be stated that in tests of fine aggregate, that is, material passing the No. 4 sieve, no difficulty due to failure by thermal volume change has been found.

The mason jar test is being used in studies of the effect of an admixture of a finely divided siliceous material on the alkali-aggregate reaction. In a preliminary investigation, concerned primarily with the amount of this admixture to be used in a reactive mortar, the results shown in figure 1 were obtained. This figure is presented to show that quantitative results can be obtained with this method of test. All specimens were prepared with a high-alkali cement, and all contained opal except one group of reference specimens. Each point in figure 1 represents an average of three specimens. The tests were discontinued at an age of 30 days, when all except the reference specimens had failed. With few exceptions, the three jars in each group gave consistent results. In the entire set of 21 groups containing reactive aggregate, the three jars in each of 14 groups broke at the same age, or at an age of within 1 day of the average for the three jars. In only four groups did any of the jars break at an age more than 2 days from the average. As all jars developed the same type of cracking, it is believed that a marked difference between the ages of failure of specimens of a group may be due to differences in strength between the jars. Methods for classifying jars prior to use are being considered.

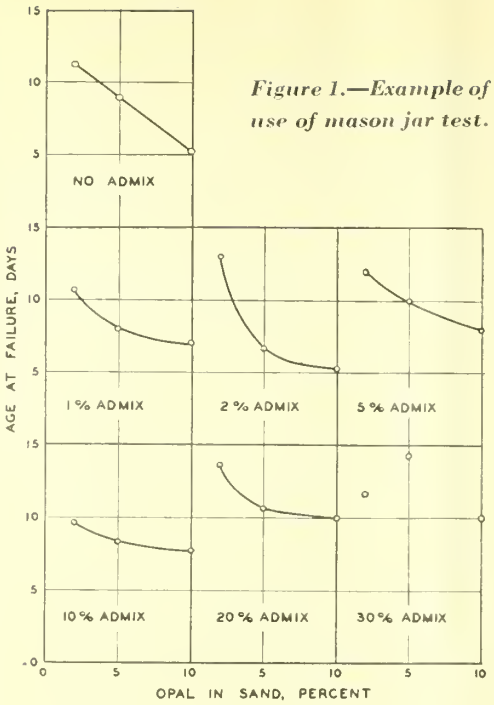

Figure 1.—Example of use of mason jar test.

The test is considered of most value for the rapid determination of the susceptibility of aggregates to alkali, and for the determination of the compatibility of given combinations of cement and aggregates. The test specimens are prepared readily with very simple equipment. Although a balance has been used to weigh the components of the mortar or concrete, proportioning of the mix by volume should be satisfactory. For 1:2 mortars, a water-cement ratio of 0.5 by weight, or 0.75 by volume, has been found suitable. When it is desired to test concrete mixtures, sufficient water to give a plastic mix should be used. The prepared mortar or concrete is placed in the coated jar, with care to prevent scratching the coating, and the bottom of the jar struck several times with the palm of the hand to settle the contents. The jar is then capped. At an age of 20 to 24 hours, about 25 ml. of water is added to the jar, the cap replaced firmly, and the jar placed in storage. Daily inspection of the jars is advisable. When the jars are stored in an inverted position, the cracking frequently can be seen without moving the jar if the bottom and an inch or so of the side of the jar can be inspected.

Various forms of cracking are found. Sometimes a simple vertical crack will appear. If this cracking does not develop further, it is considered to indicate an expansion of very low order, resulting possibly from an alkali-aggregate reaction which may be dismissed as immaterial. However, if simple cracking progresses into multiple cracking or, as is most often the case, the jar develops multiple cracking at the initial failure, the failure may be considered to indicate an aggregate of high susceptibility to reaction with the alkali in cement. Sketches of jars showing simple cracking, simple cracking which developed into multiple cracking, and initial multiple cracking are shown in figure 2.

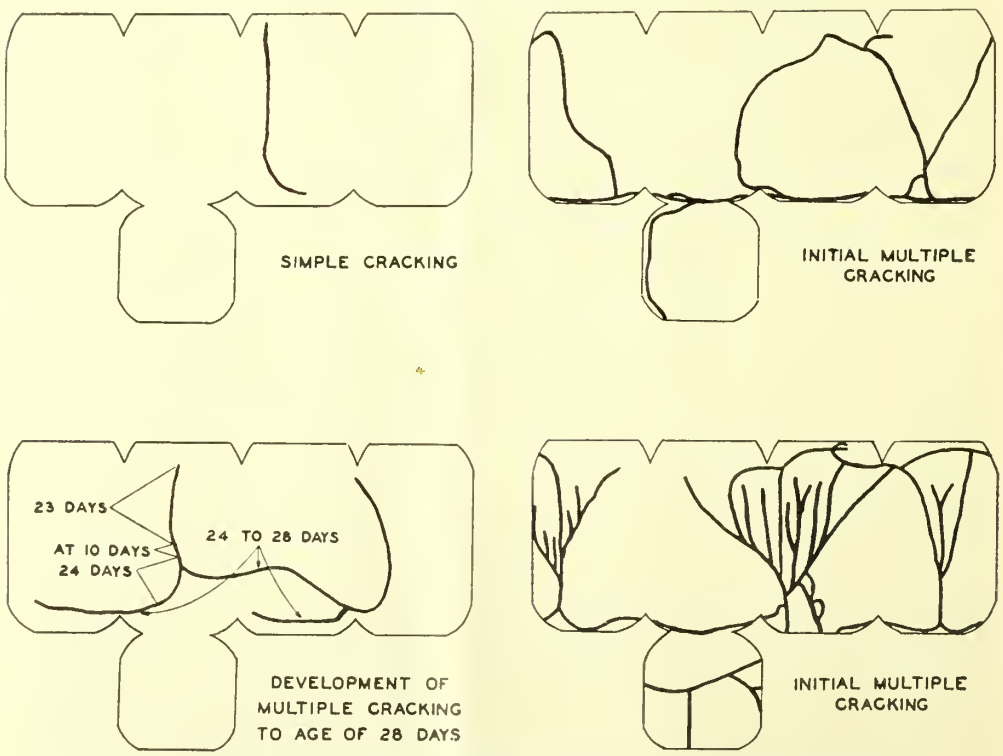

Figure 2.—Failures of mason jars (developed views of sides and bottom of jars).

Conducting Long-Range Highway Needs Studies

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH
PUBLIC ROADS ADMINISTRATION

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The great upsurge of motor-vehicle travel since the end of the war has directed public attention to the rapidly mounting inadequacy of our road and street systems. It was realized years ago that complete appraisal of the entire field of highway transportation was needed, and the highway planning surveys, undertaken in the middle 1930's, were conceived for that purpose.

The preparation of all-inclusive, long-range plans is increasingly urgent. Several States have recently completed or have under way the preparation of comprehensive reports on the engineering, financial, and administrative phases of highway needs. These reports, which are actually the expected culmination of the State highway planning surveys, are usually prepared under the auspices of a specially created highway study committee of the State legislature, and are conducted in cooperation with each level of government having jurisdiction over highways, roads, or streets. The cooperation of other groups interested in the highway program, such as the motor-vehicle user groups, local civic associations, and farm and business interests, is likewise essential.

Preparation of the reports of highway needs requires consolidation of all available factual data, most of which is at hand in the planning surveys; collection of additional data needed to round out the portrayal of the current situation; and recommendation of the program for future action. Up to the present time, the engineering phases of the long-range reports have imposed the greatest demand upon the time, talent, and ingenuity of the highway engineers in those States that have undertaken them. Accordingly, the following discussion relates principally to the means by which the preparation of the engineering report is placed under way and some of the more unusual problems encountered in the conduct of this phase of a long-range highway needs study.

RAPID INCREASES in motor-vehicle ownership and travel that have developed since the end of World War II are bringing into focus the major weaknesses of the Nation's highways, roads, and streets—obsolescence, deterioration, and congestion, just to mention a few. New problems in providing adequate facilities have arisen from significant population shifts, from changes in land usage, and from other factors such as increased mechanization and development of new industries during and after the war. The backlog of deferred work that accumulated during the war; the present shortages of labor, equipment, materials, and transportation; and the current price situation have created additional problems.

It was realized, years ago, that a complete appraisal of the entire field of highway transportation, and its relation to other forms of transportation, was urgently needed. The highway planning surveys, undertaken in the middle 1930's by the State highway departments in cooperation with the Public Roads Administration, had that goal for their purpose. These surveys were conceived as instruments for the collection, analysis, interpretation, and summarization of complete factual data relating to the construction, maintenance, operation, and administration of all branches of the highway plant. Their ultimate aim, and the primary one, was the preparation of a documented, comprehensive report concerning adequate highway systems, and means for their fulfillment.

Over the past 3 years the need for such all-inclusive, long-range plans has become increasingly apparent. It has become evident that prewar policies, necessarily of restricted scope because they were largely dictated by the limited revenues available for expenditure, would be wholly inadequate to cope with mounting highway [1] needs. Demands for improvement come from all sides, for all classes of highways, from the lightly traveled country road to the heavily traveled city thoroughfare.

THREE PHASES OF STUDY

Out of the present situation has come an urgent interest, on the part of an increasing number of States, in the intensive studies of long-range needs—the studies which were, more than 10 years ago, set forth as the goal of the highway planning surveys.

These long-range needs studies, as proposed when the planning surveys were first initiated—proposals still valid and timely—can be divided logically into three phases:

The engineering study.—This involves an engineering appraisal of the improvements needed and costs involved in bringing the highways up to required standards.

The finance and taxation study.—This includes studies of methods of financing the program, determinations of responsibility of support for various classes of roads and streets, degree of support by classes of users, etc.

[1] Throughout this report all classes of highways, roads, and streets are frequently referred to simply as "highways."

The administrative study.—This includes appraisal of the administrative relationships and controls necessary for orderly prosecution of the program and of the organizational structures and operating practices of various governmental units having jurisdiction over highways.

Work in connection with these phases can be conducted simultaneously and completed at about the same time, but whether it is preferable to issue the reports separately or in combination is a matter to be decided in each particular instance.

There are, of course, considerable overlapping interests between each of the three categories. Up to the present time, the engineering phases of the long-range reports have imposed the greatest demand upon the time, services, talent, and ingenuity of the highway engineers in those States which have undertaken such comprehensive studies. Because of this circumstance, and because a sound engineering appraisal of highway needs is an absolute and basic essential to any study of financing and administration, the following discussions will relate principally to the engineering phases of the long-range highway needs reports.

FIRST REPORTS COMPLETED

The first of the comprehensive engineering reports as contemplated by the planning surveys, covering all highways in a State, were *Engineering Facts and a Future Program* prepared for the California legislature in 1946 and *Highway Needs in Michigan* prepared by the Michigan Good Roads Federation in 1947. Other States have recently completed or have under way the preparation of engineering reports following somewhat the same general pattern. It should be recognized that improvements in the method of approach and presentation of results are continually being made and that there is no fixed step-by-step procedure to follow. Each study must be conducted in a fashion that conforms most nearly to the requirements for the particular State.

The elements of a long-range program are such that they cannot be handled successfully when limited to a single class of roads or to a single jurisdictional unit. The long-range program must be developed on a broad base, and experience has demonstrated that it must embrace every mile of highway, road, and street in the State.

Further, the success of the long-range program depends upon the support given by

various levels of government—State, counties, cities, towns, townships, and boroughs—and by various other groups interested in the highway program, such as the motor-vehicle user groups, municipal and county leagues or associations, farm and business interests, the construction industry, equipment manufacturers, and materials producers. Each of these must participate to the maximum extent to insure that the long-range program is soundly conceived and executed and in the best interests of all groups concerned and of the public at large.

SPONSORSHIP OF STUDY

There are various means of sponsoring the preparation of a long-range highway program. Among these are (1) a committee created by the State legislature, (2) a committee appointed by the Governor, and (3) a committee established by a citizens' association or a road-user group. In the latter two cases, it is desirable that the legislature be involved at least to the extent of creating an advisory committee to work with the sponsoring committee.

From an engineering standpoint, the State highway department is admittedly the organization best qualified to conduct the comprehensive study of highway needs in a particular State. Hence, in some instances, the legislature or the Governor may direct the State highway department to make the entire study. This procedure is not usually employed, however, because of the numerous public relations problems involved. By its very nature, a governmental agency is limited in the scope of the problems it can handle without suspicion of bias, especially in fields into which its activities and jurisdiction do not normally extend.

Because of the need for dealing with agencies at various levels of government, and with the many public and private groups having varying degrees of interest in the numerous problems affecting the highway program, it appears preferable in most instances to conduct a long-range highway needs study under a legislative mandate by means of a specially created committee composed of representatives of all parties concerned. If, because of the unwieldiness of a large group, it is not practicable to afford complete representation to all parties on the parent committee, advisory subcommittees can be created to enable their active participation.

For reasons similar to those previously cited, the parent committee will ordinarily prefer to have the study headed by an outside specialist in this field rather than by a member of the State highway department. In this connection, however, it may be pointed out that the talent currently available and qualified to organize and carry to satisfactory completion an engineering study of this magnitude is extremely limited.

There are various means of financing these studies, utilizing State and local funds, contributions by citizens' and user groups, and Federal funds. The Public Roads Administration has given full endorsement to the purposes of these surveys and has expressed its desire to render all possible aid in their execution. The conditions for obtaining Federal funds to aid in paying the expenses of studies of this kind are set forth in a statement of the Public Roads Administration dated September 24, 1947.

The fitting together of the complex and voluminous array of factual information into a long-range program is an undertaking of considerable magnitude. Intensive effort is required to organize a study, place it under way, and complete it within a reasonable period. Among the most valuable functions performed by the specialist, hereafter referred to as the engineer-director, is the step-by-step education of the committee membership. Committee members usually do not have an engineering background; but by holding periodic meetings at which the engineer-director can report progress and discuss the significance and interpretation of the findings which are being obtained, immeasurably beneficial results are realized. The county representatives become familiar with the problems in the cities and on the main highways, the cities acquire a greater appreciation of the rural road problem, and various special interests have the opportunity for round-table discussion of their own and each other's viewpoints. Broadening of outlook and a greater awareness and appreciation of the complex problems involved in highway administration, planning, programming, design, construction, and maintenance soon become evident. Artificial barriers are broken down in a mutual effort to solve highway problems equitably and in the best interests of all concerned.

PUBLIC HEARINGS

Public hearings conducted by the legislative committee are a customary and valuable phase of making a long-range highway study. At such hearings, usually held at a number of places through the State, the public has the opportunity to become informed about the study and to offer opinions on the nature of the road problems in their area. The timing of these public hearings is most important. There is a general tendency to hold such hearings too early, prior to starting the field work, when there is little in the nature of concrete proposals or procedures that can be presented to the public. As a result, public enthusiasm will gradually recede and the final report may come as an anticlimax.

On the other hand, public hearings can be of tremendous value in stimulating local cooperation when they are held at the time the counties, cities, and towns are contacted with respect to making an engineering appraisal of their own particular road and street needs. Since a high degree of public interest is desirable at the time of issuance of the report, public hearings are also of value when held just in advance of the date of issuance. Possibly the best arrangement is for some hearings to be held at both times.

The time schedule for undertaking a report should include allowances of about 3 months for completion of the preliminary arrangements and about 12 months additional for the active work program. If the study is scheduled for a shorter time, the work load is likely to become too heavy and certain desirable features may be sacrificed; if scheduled over a longer period, some of the phases become stale, interest in the work subsides, and very little practical improvement in the quality of the report is obtained. It is recognized, of course, that unusual complexity or magnitude of the work may make it desirable to conduct the study over a longer period in a particular State. In some instances, also, personnel shortages and cost considerations may be sufficient reason for scheduling the study over a longer period.

PRELIMINARY OPERATIONS

Essential preliminary operations in undertaking a highway needs study are the preparation of a report outline and a work program schedule. The complete report outline embraces the following:

1. Introduction.
2. History of highway development in the State.
3. Characteristics of road use.
4. Future development of the State and its subdivisions.
5. Classification of road systems.
6. Standards of road improvement.
7. Standards of road operational improvement (traffic control, lighting, parking, law enforcement, and maintenance standards).
8. Elements of road cost.
9. Future road improvement program.
10. Finance and taxation.
11. Highway administration.
12. Highway vehicle and traffic regulation.
13. Conclusions.

These items are the main headings included in a suggested outline set forth in a statement of the Public Roads Administration dated September 24, 1947, in which there is a listing of the detailed items included under each major heading. A similar discussion had been prepared for the highway planning surveys some years earlier. The main elements of the outline are applicable to practically all States. The emphasis to be placed upon various segments, however, will necessarily vary from State to State, depending upon the nature of the particular highway problems in each State.

PLANNING SURVEYS PROVIDE DATA

The major source of information will, in practically all States, be the State highway department's highway planning survey. Generally there is a wealth of information relating to the State highway system but only limited data relating to roads and streets beyond the jurisdiction of the State highway department. Actually, it had been anticipated that the planning surveys would collect data on all highways but many States, limited in their resources, spent their principal efforts on the State highway systems. This situation, which is quite common, requires careful planning of the techniques and procedures to be employed in evaluating the highway needs for all

jurisdictions on a comparable, straightforward, and easily understandable basis.

Probably the greatest single problem in a study of highway needs, from the standpoint of planning, time, effort, and coverage, is the work that must be done by the various jurisdictional units in connection with determining the costs of their respective future road improvement programs. A work program for this phase of the study involves the following step-by-step operations:

1. Field classification of roads and streets into logical systems.
2. Field appraisal of needs for improvement.
3. Processing and analysis of collected data.

MANUALS OF INSTRUCTION

The work in connection with the first two items is usually conducted in conformity with procedures outlined in manuals of instruction prepared by the staff of the study committee specifically for each administrative jurisdiction. Thus, in a State where there are three jurisdictions (State highway department, counties, and cities) for the construction and maintenance of highways, there would be three corresponding series of manuals of instruction.

The principal variations in these manuals would be those occasioned by the differences in the character of the road systems and the differences in the caliber of the respective engineering organizations at the various governmental levels. For example, very rigid criteria can be applied to State highway needs, since the State highway departments have the background of experience and research as well as the engineering talent needed to undertake a comprehensive study. Most counties and cities, however, are not correspondingly situated, and accordingly the criteria employed must be on a broader base to enable a reasonably defensible appraisal of needs to be made by the county and city personnel.

PARTICIPATION NEEDED

The success in gaining acceptance of the final results of a long-range engineering study depends in large measure on the extent to which each governmental unit having any degree of responsibility for the construction and maintenance of roads or streets participates in the evaluation of the needs for those roads or streets under its jurisdiction. Thus, in those cases where no qualified engineering personnel are available at the county or city levels, it may be necessary to hire private consultants or to depend upon the State highway department personnel to do the work. In general, this latter procedure should be considered only when the counties or cities are wholly unable to do the work themselves. Even in such instances every effort should be made to have a local representative collaborate with the State highway department personnel at the time the classification and estimates of needs for a particular county or city are being appraised. In this fashion, each county and city will, to the maximum extent possible, be an active participant in the work.

SYSTEM CLASSIFICATION

With respect to the work program, the field classification of roads and streets into systems should be made independently and in advance of the field appraisal of needs. This classification is simply the sorting of existing roads and streets into groups on the basis of function, taking into account such factors as integration, predominant purpose of use, suitability from the standpoint of equitable financing, and adaptability to efficient and economical administration by various levels of government.

The classification, together with an inventory of existing mileages by types, is prepared by the State highway department, the counties, and the cities, in accordance with procedures prescribed in manuals of instruction, for the roads and streets currently under their respective jurisdictions, and is submitted to the engineer-director of the highway study committee. The work is then reviewed by the staff of the engineer-director, and errors and differences in interpretation are ironed out in each individual case.

NEEDS APPRAISAL

As this work nears completion, the field appraisal of needs can be undertaken. This involves determination of existing deficiencies and estimating of costs of needed improvements for every mile of highway. This phase of the work is also governed largely by manuals of instruction prepared for the purpose. In these manuals, it is the general practice to prescribe two sets of standards relating to widths, surface type, alinement, grades, etc.: (1) Tolerable standards, and (2) construction standards. Tolerable standards are those which are considered satisfactory or tolerable on highways now in service. Any road section or other facility that does not measure up to the tolerable standard is considered eligible for improvement. In some cases it may be questionable whether a single particular deficiency is of sufficient importance to warrant an improvement. In other cases this same deficiency, in combination with other deficiencies, would indicate the immediate need for improvement. If it is determined that an improvement is warranted, as measured by the tolerable standards, the improvement to be planned should be in accordance with the construction standards, and the costs estimated accordingly.

In a given situation it might be assumed, for example, that for surfaced width for a rural road carrying a certain traffic volume the tolerable standard is 18 feet and the construction standard is 20 feet (minimum) to 24 feet (maximum).

If the existing road in question has a surfaced width of 18 feet or more, it would not be considered deficient in width. If, however, the surface is only 16 feet wide, it would be considered deficient in this respect. When the manner of overcoming this particular deficiency is being considered, it would be necessary to take into account all service factors—terrain, volume and character of traffic, etc.—in order to determine whether the proposed improvement should be constructed to 20 feet in width, 24 feet in width, or somewhere in-between. Further, all deficiencies must be considered in total for any given road section when arriving at the final estimate of cost for a proposed improvement. This procedure summarizes, quite briefly, the method by which selected standards are applied to existing highways to determine the extent of existing deficiencies.

As the reports upon existing deficiencies and estimates of cost of needed improvements are prepared and submitted to the engineer-director by the State highway department, the counties, and the cities, they are reviewed for consistency, adherence to instructions, completeness, and accuracy. This is generally termed the "screening" process and is one of the most important phases of the entire engineering study. Most counties and cities can be depended upon to prepare their estimates in accordance with the instructions. Invariably, however, some will submit estimates that are obviously too low, too high, or incomplete. Such estimates must be taken up again with the reporting agency, and all errors, omissions, and differences in interpretation ironed out in each case.

Experience indicates that from 6 to 8 months should be allowed for the field work in connection with classification and appraisal of needs. The time required depends upon such factors as the number of governmental units having jurisdiction over highways; the caliber of the various engineering organizations in each jurisdiction; the degree of cooperation generated among the various levels of government; and the number and caliber of personnel made available as a staff to the engineer-director.

PROCESSING AND ANALYSIS

When the screening of the field reports of deficiencies and costs is completed, the work of processing and analyzing the reports is commenced. This phase of the work can be relatively simple or it can be complicated and confused, depending upon the amount of advance study given to each previous operation.

For example, it must be decided at the very outset of the engineering study whether manual or tabulating machine methods are to be employed in the final processing. On the basis of experience to date it can be stated that, almost without exception, tabulating machine methods are far superior to manual methods for this type of study. Accordingly, the following discussion relates almost entirely to tabulating machine methods, although the same principles will, of course, apply to manual methods.

It is essential, prior to starting any field work, that there be a clear understanding of what data are needed, how the data are to be obtained, and how the data are to be analyzed. This requires intensive preliminary study. To aid in this phase of the work, arrangements should be made for consultation with the State highway department personnel, county and city representatives, Public Roads Administration technicians, and experts in other public

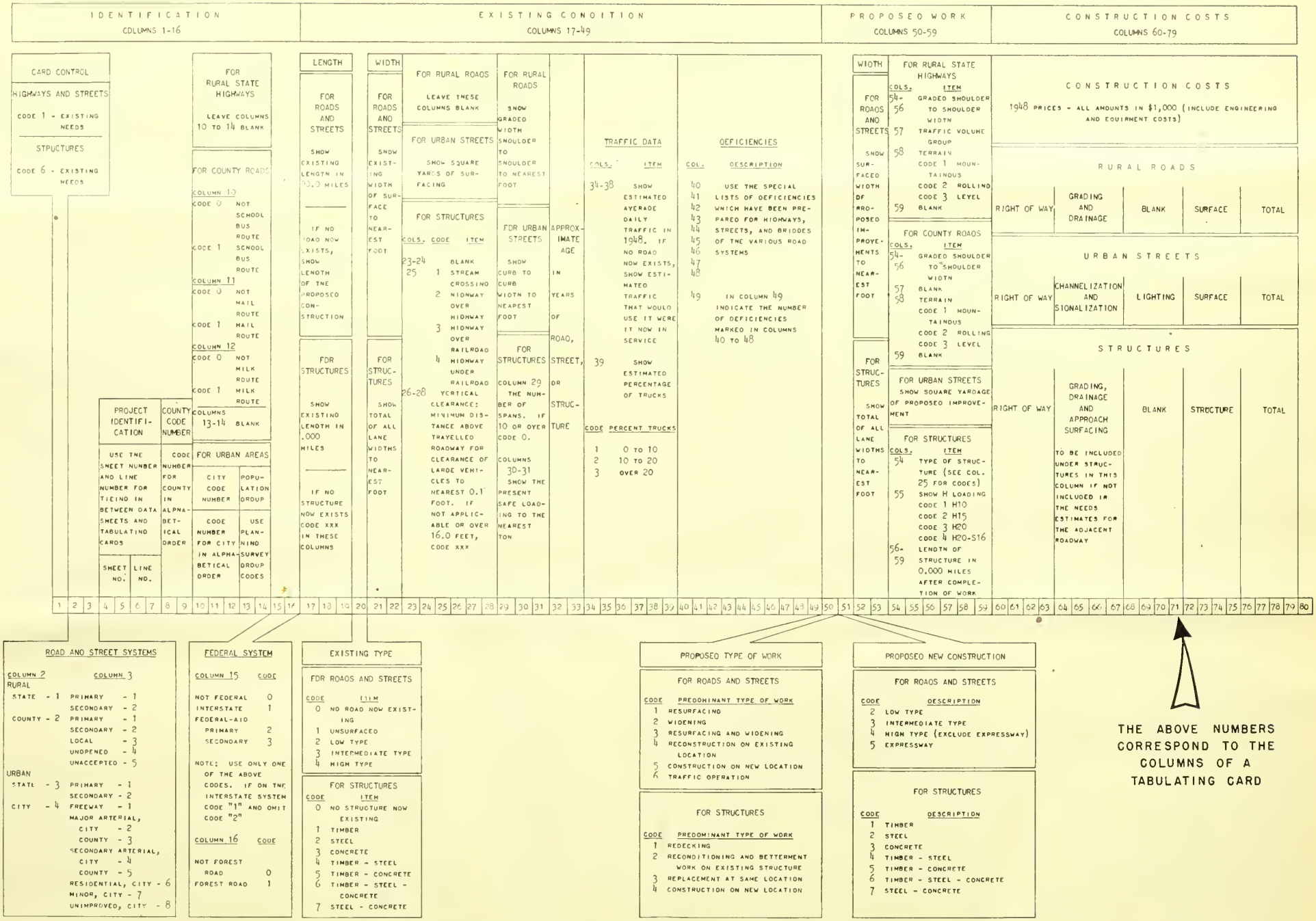
IDENTIFICATION
COLUMNS 1-16
EXISTING CONDITION
COLUMNS 17-49
PROPOSED WORK
COLUMNS 50-59
CONSTRUCTION COSTS
COLUMNS 60-79
CARD CONTROL
HIGHWAYS AND STREETS
CODE 1 - EXISTING NEEDS
STRUCTURES
CODE 6 - EXISTING NEEDS
PROJECT IDENTIFICATION
USE THE SHEET NUMBER AND LINE NUMBER FOR TIEING IN BETWEEN DATA SHEETS AND TABULATING CARDS
SHEET NO.
LINE NO.
COUNTY CODE NUMBER
CODE NUMBER FOR COUNTY IN ALPHABETICAL ORDER
FOR RURAL STATE HIGHWAYS
LEAVE COLUMNS 10 TO 14 BLANK
FOR COUNTY ROADS
COLUMN 10
CODE 0 NOT SCHOOL BUS ROUTE
CODE 1 SCHOOL BUS ROUTE
COLUMN 11
CODE 0 NOT MAIL ROUTE
CODE 1 MAIL ROUTE
COLUMN 12
CODE 0 NOT MILK ROUTE
CODE 1 MILK ROUTE
COLUMNS 13-14 BLANK
FOR URBAN AREAS
CITY CODE NUMBER
POPULATION GROUP
CODE NUMBER FOR CITY IN ALPHABETICAL ORDER
USE PLANNING SURVEY GROUP CODES
LENGTH
FOR ROADS AND STREETS
SHOW EXISTING LENGTH IN 0.0 MILES
IF NO ROAD NOW EXISTS, SHOW LENGTH OF THE PROPOSED CONSTRUCTION
FOR STRUCTURES
SHOW EXISTING LENGTH IN .000 MILES
IF NO STRUCTURE NOW EXISTS CODE XXX IN THESE COLUMNS
WIDTH
FOR ROADS AND STREETS
SHOW EXISTING WIDTH OF SURFACE TO NEAREST FOOT
FOR STRUCTURES
SHOW TOTAL OF ALL LANE WIDTHS TO NEAREST FOOT
FOR RURAL ROADS
LEAVE THESE COLUMNS BLANK
FOR URBAN STREETS
SHOW SQUARE YARDS OF SURFACING
FOR STRUCTURES
COLS. CODE ITEM
23-24 25 BLANK
1 STREAM CROSSING
2 HIGHWAY OVER HIGHWAY
3 HIGHWAY OVER RAILROAD
4 HIGHWAY UNDER RAILROAD
26-28 VERTICAL CLEARANCE: MINIMUM DISTANCE ABOVE TRAVELLED ROADWAY FOR CLEARANCE OF LARGE VEHICLES TO NEAREST 0.1 FOOT. IF NOT APPLICABLE OR OVER 16.0 FEET, CODE XXX
FOR RURAL ROADS
SHOW GRADED WIDTH SHOULDER TO SHOULDER TO NEAREST FOOT
FOR URBAN STREETS
SHOW CURB TO CURB WIDTH TO NEAREST FOOT
FOR STRUCTURES
COLUMN 29 THE NUMBER OF SPANS. IF 10 OR OVER CODE 0.
COLUMNS 30-31 SHOW THE PRESENT SAFE LOADING TO THE NEAREST TON
APPROXIMATE AGE IN YEARS OF ROAD, STREET, OR STRUCTURE
TRAFFIC DATA
COLS. ITEM
34-38 SHOW ESTIMATED AVERAGE DAILY TRAFFIC IN 1948. IF NO ROAD NOW EXISTS, SHOW ESTIMATED TRAFFIC THAT WOULD USE IT WERE IT NOW IN SERVICE
39 SHOW ESTIMATED PERCENTAGE OF TRUCKS
CODE PERCENT TRUCKS
1 0 TO 10
2 10 TO 20
3 OVER 20
DEFICIENCIES
COL. DESCRIPTION
40 41 42 43 44 45 46 47 48 USE THE SPECIAL LISTS OF DEFICIENCIES WHICH HAVE BEEN PREPARED FOR HIGHWAYS, STREETS, AND BRIDGES OF THE VARIOUS ROAD SYSTEMS
49 IN COLUMN 49 INDICATE THE NUMBER OF DEFICIENCIES MARKED IN COLUMNS 40 TO 48
WIDTH
FOR ROADS AND STREETS
SHOW SURFACED WIDTH OF PROPOSED IMPROVEMENTS TO NEAREST FOOT
FOR STRUCTURES
SHOW TOTAL OF ALL LANE WIDTHS TO NEAREST FOOT
FOR RURAL STATE HIGHWAYS
COLS. ITEM
54-56 GRADED SHOULDER TO SHOULDER WIDTH
57 TRAFFIC VOLUME GROUP
58 TERRAIN
CODE 1 MOUNTAINOUS
CODE 2 ROLLING
CODE 3 LEVEL
59 BLANK
FOR COUNTY ROADS
COLS. ITEM
54-56 GRADED SHOULDER TO SHOULDER WIDTH
57 58 BLANK
TERRAIN
CODE 1 MOUNTAINOUS
CODE 2 ROLLING
CODE 3 LEVEL
59 BLANK
FOR URBAN STREETS
SHOW SQUARE YARDAGE OF PROPOSED IMPROVEMENT
FOR STRUCTURES
COLS. ITEM
54 TYPE OF STRUCTURE (SEE COL. 25 FOR CODES)
55 SHOW H LOADING
CODE 1 H10
CODE 2 H15
CODE 3 H20
CODE 4 H20-S16
56-59 LENGTH OF STRUCTURE IN 0.000 MILES AFTER COMPLETION OF WORK
CONSTRUCTION COSTS
1948 PRICES - ALL AMOUNTS IN $1,000 (INCLUDE ENGINEERING AND EQUIPMENT COSTS)
RURAL ROADS
RIGHT OF WAY
GRADING AND DRAINAGE
BLANK
SURFACE
TOTAL
URBAN STREETS
RIGHT OF WAY
CHANNELIZATION AND SIGNALIZATION
LIGHTING
SURFACE
TOTAL
STRUCTURES
RIGHT OF WAY
GRADING, DRAINAGE AND APPROACH SURFACING
TO BE INCLUDED UNDER STRUCTURES IN THIS COLUMN IF NOT INCLUDED IN THE NEEDS ESTIMATES FOR THE ADJACENT ROADWAY
BLANK
STRUCTURE
TOTAL
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74 75 76 77 78 79 80
ROAD AND STREET SYSTEMS
COLUMN 2 COLUMN 3
RURAL
STATE - 1 PRIMARY - 1
SECONDARY - 2
COUNTY - 2 PRIMARY - 1
SECONDARY - 2
LOCAL - 3
UNOPENED - 4
UNACCEPTED - 5
URBAN
STATE - 3 PRIMARY - 1
SECONDARY - 2
CITY - 4 FREEWAY - 1
MAJOR ARTERIAL, CITY - 2
COUNTY - 3
SECONDARY ARTERIAL, CITY - 4
COUNTY - 5
RESIDENTIAL, CITY - 6
MINOR, CITY - 7
UNIMPROVED, CITY - 8
FEDERAL SYSTEM
COLUMN 15 CODE
NOT FEDERAL 0
INTERSTATE 1
FEDERAL-AID PRIMARY 2
SECONDARY 3
NOTE: USE ONLY ONE OF THE ABOVE CODES. IF ON THE INTERSTATE SYSTEM CODE "1" AND OMIT CODE "2"
COLUMN 16 CODE
NOT FOREST ROAD 0
FOREST ROAD 1
EXISTING TYPE
FOR ROADS AND STREETS
CODE ITEM
0 NO ROAD NOW EXISTING
1 UNSURFACED
2 LOW TYPE
3 INTERMEDIATE TYPE
4 HIGH TYPE
FOR STRUCTURES
CODE ITEM
0 NO STRUCTURE NOW EXISTING
1 TIMBER
2 STEEL
3 CONCRETE
4 TIMBER - STEEL
5 TIMBER - CONCRETE
6 TIMBER - STEEL - CONCRETE
7 STEEL - CONCRETE
PROPOSED TYPE OF WORK
FOR ROADS AND STREETS
CODE PREDOMINANT TYPE OF WORK
1 RESURFACING
2 WIDENING
3 RESURFACING AND WIDENING
4 RECONSTRUCTION ON EXISTING LOCATION
5 CONSTRUCTION ON NEW LOCATION
6 TRAFFIC OPERATION
FOR STRUCTURES
CODE PREDOMINANT TYPE OF WORK
1 REDECKING
2 RECONDITIONING AND BETTERMENT WORK ON EXISTING STRUCTURE
3 REPLACEMENT AT SAME LOCATION
4 CONSTRUCTION ON NEW LOCATION
PROPOSED NEW CONSTRUCTION
FOR ROADS AND STREETS
CODE DESCRIPTION
2 LOW TYPE
3 INTERMEDIATE TYPE
4 HIGH TYPE (EXCLUDE EXPRESSWAY)
5 EXPRESSWAY
FOR STRUCTURES
CODE DESCRIPTION
1 TIMBER
2 STEEL
3 CONCRETE
4 TIMBER - STEEL
5 TIMBER - CONCRETE
6 TIMBER - STEEL - CONCRETE
7 STEEL - CONCRETE
THE ABOVE NUMBERS CORRESPOND TO THE COLUMNS OF A TABULATING CARD

Figure 1.—Highway needs master control sheet for form set-up and machine tabulations of rural and urban road, street, and structure data.

or private organizations having prior experience or outstanding qualifications in studies of this type.

One major objective of such preliminary study and consultation is the development of a master control sheet showing the nature and sequence of the items to be processed in the various operations leading up to the final tabulations of the needs data. Figure 1 shows a sample lay-out for a master control sheet to which are geared the mechanics of the following phases of the engineering study: (1) Inventory of roads, streets, and bridges; (2) preparation of manuals of instruction for appraisal of needs; (3) design of record forms; (4) preparation of coding manual; (5) design of code sheets and tabulating cards; and (6) lay-out of final tabulations.

Once the engineer-director has approved the master control sheet, there should be no changes except with his specific approval. The insertion of additional data or a rearrangement of items for city streets, for example, could readily complicate the coding sequence and even require special attention to tabulating machine operations to an extent that would nullify the advantages inherent in the basic master control sheet set-up.

Figure 2 shows a work sheet used for recording deficiencies and cost of needed improvements for county roads. It will be noted that the sequence and arrangement of items on this form correspond to those on the master control sheet (fig. 1). Other forms, such as those for State highways and city streets, also correspond to the master control sheet. Figure 3 represents column headings for a series of six code sheets covering roads and structures on city streets, county roads, and State highways. These headings likewise follow the arrangement of the master control sheet.

TABULATION CONTROLS

The forms illustrated in figures 1, 2, and 3 are practically identical with those used in a State that has recently completed an engineering study, and represent a pioneer effort in setting up fixed controls for the mechanics of conducting the work. Since it is the primary purpose here to indicate the general nature of the master control sheet, there is no discussion included in this report of the many details of its design. Such details must be worked out for each individual State.

At every step of the processing operations for each county and for each city, care must be exercised to maintain frequent manual controls, especially for mileages and costs. Since there is such a mass of data being handled in a relatively short period of time, usually by inexperienced help, these manual controls are necessary in order that any missing or duplicated data can be readily spotted at any stage of the operations. These controls are prepared for the "as submitted" programs prior to screening, and for the "accepted" programs after completion of screening, and are also of tremendous value in proving the final tabulating machine listings.

After the tabulating cards have been proved, the final summaries of deficiencies, mileages, and costs in the needs program can be prepared. There is no particular problem involved in making the necessary tabulations, since they can be obtained readily from the punch cards. There are, however, certain tabulations relating to mileages and costs that are essential as matter of continuous reference and permanent record. These tabulations show the relationship between existing surface types, kind of work recommended, new type of surface, mileage, and costs. A typical arrangement of the form used for summarizing these data for rural roads is shown in figure 4. Similar tables should be prepared for structures and for city streets.

EXISTING CONDITION

5 LENGTH (IN 0.0 MILES)

6 TYPE SURFACE

7 SURFACE WIOTH

8 ROADWAY WIDTH

9 APPROXIMATE AGE DF SURFACE

10 ESTIMATED DAILY TRAFFIC

11 TRUCKS AND BUSSES
0-10% ... ☐ 10-20% ... ☐ OVER 20% .... ☐

IDENTIFICATION

1 COUNTY

2 ROAD NAME, NUMBER AND/OR LOCATION

3 PROJECT NUMBER

4 SYSTEM
C-1 .. ☐ SCHOOL BUS ROUTE ..... ☐ FEDERAL AID SECONDARY .. ☐
C-2 .. ☐ U.S. MAIL ROUTE ..... ☐ U.S. FOREST HIGHWAY .... ☐
C-3 .. ☐ MILK COLLECTION ROUTE .. ☐ OTHER FEDERAL .... ☐
C-4 .. ☐

DEFICIENCIES

	EXISTING	TOLERANCE	DEFICIENT
12 TYPE SURFACE			☐
13 SURFACE WIDTH			☐
14 ROADWAY WIDTH			☐
15 GRADIENT			☐
16 CURVATURE			☐

DEFICIENT

17 ROADWAY BASE ☐
18 DRAINAGE ☐
19 SURFACE CONDITION ☐
20 SPECIAL WARRANTS * ☐
* (EXPLAIN IN REMARKS)

PROPOSED WORK

TYPE OF WORK

21 RESURFACING ☐
22 WIDENING SHOULDERS ☐
23 RESURFACING AND WIDENING ☐
24 RECONSTRUCTION ON SAME ALINEMENT ☐
25 CONSTRUCTION ON NEW LOCATION ☐

31 MOUNTAINOUS .. ☐ ROLLING .. ☐ LEVEL .. ☐

STANDARD OF CONSTRUCTION

26 TYPE SURFACE _____
27 SURFACE WIDTH _____
28 ROADWAY WIDTH _____
29 MAXIMUM GRADE _____
30 MAXIMUM CURVATURE _____

COSTS

32 RIGHT-OF-WAY ..... $ _____
33 GRADE AND DRAIN ..... $ _____
34 SURFACING ..... $ _____
TOTAL ..... $ _____

NOTE: INCLUDE PRORATED ENGINEERING COSTS IN ABOVE ESTIMATES. USE 1948 PRICES.

REMARKS
(EXPLAIN SPECIAL CONDITIONS SUCH AS TIME DELAYS, ACCIDENT HAZARDS, DUST, SNOW, ICE, ETC.)

Figure 2.—Project work sheet for county road needs.

SCHEDULING PROGRAMS

With the data at hand relating to the existing deficiencies and costs of needed improvements, the next step is the scheduling of the program. It cannot, within reason, be expected that existing deficiencies can be overcome in 1 or 2 years. Within the limits of foreseeable revenues, any period up to as much as 20 years may be required for bringing the highway systems up to a satisfactory standard of improvement. It has been the usual practice in earlier studies of this nature to present the total costs of the program for a given period of years, say for a 5- or a 10-year period.

Thus, the entire analysis and presentation of results are related to a preselected fixed span of years. There are serious disadvantages to this advance selection of the program period, the principal one being its inflexibility. After the report is completed, the legislature, for example, may desire to consider a shorter or a longer period, with the result that numerous

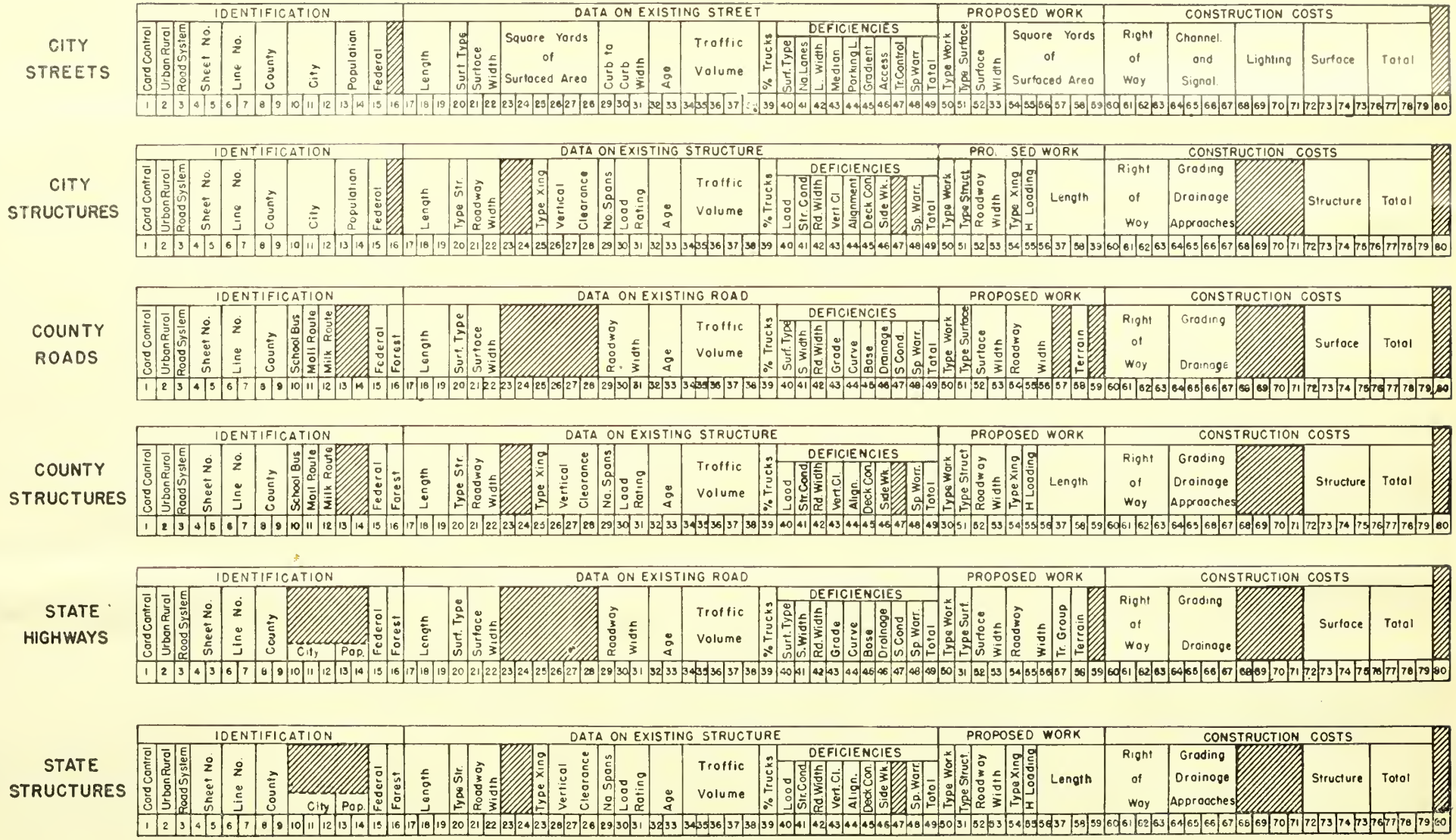

Figure 3.—Column headings for code sheets.

SERIES _____
SYSTEM _____

TYPE OF WORK	EXISTING TYPE IN SERVICE WHICH REQUIRES IMPROVEMENT	PROPOSED NEW CONSTRUCTION																					
		GRAVEL					LOW					HIGH (EXCLUDING EXPRESSWAYS)					EXPRESSWAY					TOTAL				
		MILES	COST				MILES	COST				MILES	COST				MILES	COST				MILES	COST			
			ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL
RESURFACING	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
RESURFACING AND WIDENING	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
RECONSTRUCTION ON EXISTING LOCATION	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
CONSTRUCTION ON NEW LOCATION	NONE																									
	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
TOTAL EXCLUDING WIDENING	NONE																									
	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
WIDENING	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
GRAND TOTAL																										

Figure 4.—Rural road replacements and costs summary sheet.

adjustments are made to the findings as presented in the report. Such adjustments are frequently arbitrary and sometimes involve serious errors when made by individuals not familiar with the nature or significance of the various elements of cost involved in an improvement program.

To overcome this situation, it has been the practice recently in engineering reports to present costs in a series of periods ranging from a 1-year program up to a 20-year or even a 25-year program. The very short periods and the very long periods have little significance other than to provide proper orientation and to answer certain questions which are logically raised when legislative consideration is being given to the period of years over which the highway systems should be brought up to standard.

OTHER COST ELEMENTS

In addition to the existing needs, as determined in accordance with the procedure which has been discussed, there are other cost elements involved in a long-range program that require careful analysis. These are the maintenance requirements and the additional needs accruing during the program period.

In estimating maintenance requirements, consideration is given to changes in the development of the road systems and to required improvements in standards of maintenance. Studies are made of the costs per mile or per square yard for maintaining various types of roads and streets at the stages of development which exist at the present time and which it is estimated will exist at the end of the program period. Such an analysis requires that a careful record be made of the probable mileages, by types and standard of construction, that will be constructed and retired during the period (see fig. 4). By such means, it is possible to arrive at reasonably reliable estimates of present and future maintenance requirements.

Certain additional costs, which may amount to as much as 20 percent of the total program, must be taken into account when the existing needs are spread out over any given program period in the future. For those needs that now exist but will come up for improvement in the later years of the program period, some kind of earlier stopgap improvement will be necessary to keep the affected roads in operation until the standard improvement can be made. Further, an existing road may currently be deficient only with respect to width, but by the time it comes up for improvement it may be deficient in several other respects and require complete reconstruction instead of widening alone.

It is also necessary to provide for the replacement, throughout the program period, of roads which are not in current need of improvement. In other words, it is possible that many of the existing roads are in tolerable condition as far as present-day traffic is concerned, and accordingly are not included in the program of existing needs. They will, nevertheless, deteriorate and become obsolete and require rebuilding in order to keep them up to satisfactory standards of service.

Some of the new construction built during the early years of the program period will also wear out and require rebuilding before the end of the period. The amount may be small but it must be taken into account in order that the full magnitude of the highway needs can be clearly and fully portrayed.

There is no list of factors that can be employed to evaluate these additional needs accruing during the program period. A procedure for making these estimates must be developed in each instance. Knowledge of construction practices and past experience with respect to the service life characteristics of various types of construction provides necessary background.

In addition to the needs which have already been discussed, there will be requirements for new roads, streets, and structures throughout the program period. Urban growth and redevelopment, industrial expansion, and other changes will generate new needs in the future years. Not all of these can, of course, be estimated with certainty.

ADJUSTMENT FOR PRICE LEVEL

A program of needs developed in accordance with the foregoing procedures will afford a positive index of the extent of the road and street requirements for varying program periods. At this stage of the analysis, however, the costs involved are at a price level existing at the time the estimates were made. If it is assumed, for example, that the costs of needed improvements were made at a 1948 price level, then there is the problem of adjusting the costs to the future price level that it is estimated will prevail throughout the various program periods.

It is not the purpose here to discuss or evaluate the various factors which are likely to influence the future trend of construction prices. However, a few comments concerning the nature of the problem can be made. Based

upon a prewar index of 100 percent for the 1937–41 period, the present 1948 highway construction price level is in the neighborhood of 200 percent. Let it be assumed, for example, that the index will gradually recede and level off by the mid 1950's to a certain value. From the trend thus obtained, the index for each future year can be applied to the dollar volume of work which it is expected will be accomplished during that year. This adjustment is necessary to take into account the fact that income available for expenditure can normally be expected to increase gradually throughout the program period because of increases in gasoline consumption, etc.

SINGLE INDEX USED

From a weighted price index thus obtained for each program period, the program of needs at the 1948 or any other given price level can be readily adjusted for conditions which are expected to prevail in the future. Ordinarily an index based upon the projected trend of construction prices on State highway work is considered acceptable for the purpose of adjusting maintenance costs and the programs for counties and cities. Any refinements by virtue of computing special indexes for these latter elements are difficult to justify in view of the lack of data upon which to make individual projections. Further, the small degree of apparently additional accuracy which may be involved is of dubious value in view of the lack of exactness in projecting the future price trend in the first instance.

PRIORITIES OF IMPROVEMENT

Before closing, it might be well to comment upon priorities of improvement in relation to the long-range program. Priority listings are useful in making it possible to check readily upon each highway agency with respect to progress and adherence to a predetermined program. To a considerable extent these listings are likewise useful to the various highway agencies in dealing with pressure groups. With each passing year, however, increased knowledge and experience will inevitably dictate changes and adjustments to the priority listings that will defeat the purpose for which they were originally established.

Further, the reporting forms and estimating procedures which have been discussed in connection with the engineering study are not designed for use in setting up listings of improvements in order of priority. To accomplish this latter objective would require detailed study of each proposed improvement, both individually and in relation to other improvements, to such an extent that other important phases of the long-range study would be completely overshadowed.

There is, of course, a definite place for priorities in a current 2- to 5-year program in which strict adherence to a specific schedule of improvements can be justified on the basis of comprehensive detailed analyses of all the facts available at the time the program is set up. Such a fixed and definite but relatively short-term program covering certain designated improvements should be kept continuously at hand by each highway agency. Whether a particular highway agency is in a position to actually do this depends upon the status of its planning activities and the caliber of the analyses which are brought to bear upon the problems involved.

In addition, the short-term program should be consistent with the over-all long-range highway program, which in turn must be periodically reappraised and kept up to date. This reappraisal of the long-range program should be made at least once every 10 years.

REPORT IS A BEGINNING

The completion of the long-range report must also be viewed as the real beginning, rather than the end, of the period of usefulness of the various committees which have participated in the studies. The value received by continuing the life of these or similar committees is tremendous. Being composed of informed representative citizens, they are indispensible as deliberative bodies in the field of broad administrative and financial policy relating to highway matters.

By means of these long-range highway programs, great strides are being made in the orderly development of the vast mileage of highways, roads, and streets at a rate consistent with the increasing demands of motor-vehicle traffic, and at such locations and to such standards as will result in the most economical and efficient expenditure of funds.

Manual on Uniform Traffic Control Devices for Streets and Highways

A completely new edition of the MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES FOR STREETS AND HIGHWAYS, prepared by the Joint Committee on Uniform Traffic Control Devices, has been published by the Public Roads Administration. The Manual is the generally recognized official national standard covering highway signs, pavement markings, traffic signals, islands, and similar devices used in the guidance, warning, and regulation of traffic. The importance of this work, and the need for its general adoption, will be obvious to any driver who has observed the lack of uniformity and the inadequacy of traffic control devices now used in many of our States and cities.

The Joint Committee includes representatives of the American Association of State Highway Officials, the Institute of Traffic Engineers, and the National Conference on Street and Highway Safety (now the National Committee on Uniform Traffic Laws and Ordinances). Its members are outstanding traffic engineers of State and city highway departments, the Public Roads Administration, and numerous national organizations concerned with highway use and safety. The Manual bears official approval of the three member organizations of the Joint Committee and of the American Standards Association.

Thomas H. MacDonald, Commissioner of Public Roads, has approved the Manual as implementation of section 12 of the Federal-aid Highway Act of 1944, which provides that signs, signals, and markings on roads and streets constructed with Federal aid shall be subject to approval of the State highway departments with concurrence of the Public Roads Administration. The law specifically directs the Commissioner of Public Roads to concur only in those installations that will promote the safe and efficient utilization of the highways.

Superseding editions of 1935 and 1942, the new Manual represents a 4-year effort of the Joint Committee and its staff. While basic standards generally remain unchanged, many refinements and some significant revisions have been made. The text has been entirely rewritten and expanded to provide a clearer, more complete exposition of each subject. Proper usage of traffic control devices is illustrated by 57 drawings and photographs. In addition, 85 signs are portrayed in the margins alongside the appropriate text. Many of the drawings are in color.

National standards for traffic control devices go back to 1925, when the American Association of State Highway Officials began to lay out a uniformly marked system of interstate highways. From this work grew the familiar U S numbered highway system and its well-known shield marker. In 1930 the National Conference on Street and Highway Safety adopted specifications for use of traffic control devices in cities. The two organizations created in 1931 the Joint Committee on Uniform Traffic Control Devices which produced, in 1935, the first Manual on Uniform Traffic Control Devices for Streets and Highways. The Joint Committee published a set of amendments in 1938; and in 1942 issued the War Emergency Edition, basically a condensed version of the original manual and its amendments with adaptations to wartime blackout conditions. Work on the new edition was begun in 1944 by the Joint Committee, enlarged to include the Institute of Traffic Engineers as well as the two founding organizations.

The MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES FOR STREETS AND HIGHWAYS can be purchased from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 50 cents per copy (a discount of 25 percent is allowed on orders of 100 copies or more). Payment must accompany orders. Orders should not be sent to the Public Roads Administration. No free distribution will be made.

U. S. GOVERNMENT PRINTING OFFICE: 1948

The Pentagon system, with 67 million vehicle-miles of travel a year, has an almost perfect safety record.

By THOS. H. MacDONALD,
Commissioner of Public Roads

HIGHWAY SAFETY

Driver Behavior—Key to Safe Highway Design

The Society of Automotive Engineers' Second David Beecroft Memorial Lecture

In our knowledge of the manner in which highways are used by the mass of traffic, we have reached the point at which we can coordinate driver behavior under prevailing traffic conditions with the geometric details of highway design. The degree to which the criteria so determined are accepted and intelligently applied in practice will determine the degree of safe efficiency of our future highways.

Our principally used street and highway systems are largely the product of the past third of a century. Most of the improved mileage has been built under public pressures, and also by legislative edict, to stretch the dollars over maximum lengths. In general, the design tolerances have been too meager for today's quantity and characteristics of traffic. Traffic-overloaded highways are one of the chief underlying causes of highway accidents.

Geometric highway design to induce proper driver behavior in traffic streams will require rebuilding or rehabilitation of the major mileage of both the primary and principal secondary routes to more liberal standards. The cost will exceed annual expenditures heretofore reached, and will require a long period—a minimum of 15 years and probably longer. Carefully planned annual programs of rehabilitation are an emergency need.

The appealing angle of highways designed for safe operation is that safe designs cost no more than unsafe designs, if total costs are considered. The initial cost of constructing a highway is only one element, and far less costly than other elements, of the total annual cost of highway transport. The costs of owning, maintaining, and operating more than 40 million motor vehicles are many times the expenditure for highway construction and maintenance. When these total costs to road users are considered, plus the value of lost time for the millions of passenger cars used for business and for all vehicles operated by paid drivers, and the value of saving in time on the investment of farmers, merchants, and industrialists in their vehicles and in the merchandise carried, there are few highways indeed for which the small additional investment to design for safer operation cannot be justified on the economic basis of returns to road users. When there are added to these costs the cost of highway accidents that can be prevented by highways designed for safety, the economy of such highways is unassailable. Incidentally, highway engineers have too long wrecked professional conviction against the stone wall of comparative "per mile" costs. We need a new concept that recognizes the actual passenger-mile and ton-mile services returned as the realistic measure of highway construction costs.

Highways designed for safety on the national scale will be realized all too slowly. There are many accidents which highways designed within cost possibilities cannot prevent. The only hope of maintaining the downward trend of accidents, particularly fatal accidents, is the vigorous application, enforcement, and popular support of the Action Program of the President's Highway Safety Conference.

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This well-planned highway easily and safely carries 10,000 vehicles a day at reasonable speeds.

THROUGH the many years of his activities in the rapidly expanding fields of automotive engineering and motor vehicle utilization, David Beecroft was a positive force for technical progress. He constantly brought to his associates the impression of a restless inner drive to secure action, and an eagerness, merging into impatience, to defeat delays in moving toward higher standards of excellence for the many diverse components that must be integrated and perfected to provide the desired end product—efficient highway transport. He insisted that the attack on each problem accord with sound techniques, and believed that advance comes through research. In 1921, when president of the Society of Automotive Engineers, he said: "The roots of the present are always found in the past, and we must study the past if we are to proceed correctly in interpreting sanely the days that are to come."

We might, perhaps, modestly record the progress that has been made in highway transport development since Mr. Beecroft gave expression to this principle. After four decades of accelerated growth in the extent and varieties of services provided by the motor vehicle, it is self-evident that in this period no other single factor has so profoundly affected the pattern of social and economic life in these United States. A particularly impressive chapter of the record would disclose the realized strength that highway transport provided the United States for both defense and offense during World War II.

It is more consistent with Mr. Beecroft's philosophy, however, to use the experience of the past to insure the complete rejection of the fatalistic suspicion that the current inefficiencies and hazards of highway transport are necessarily inherent.

If we are to gain mastery over highway transport as an efficient tool for national progress, and stop the toll that we are paying for its use in the tragic loss of life, the permanent injuries, and the financial costs of accidents that no nation, however wealthy, can afford, we must critically re-examine our policies and practices, particularly the thinking back of these, to find the causes that are responsible for our failures. We know we have taken many wrong turnings. Changing conditions require new horizons in our applied techniques as knowledge is unfolded and experience is more precisely interpreted. It is profitless to speculate upon what we might have done had we foreseen, in the early years, a reasonable fraction of the dimensions to which highway transport was so rapidly to grow, or had we anticipated more realistically its characteristics in actual operation.

It is a high honor to follow, in this series of papers,[1] Mr. Paul G. Hoffman, who in 1947 presented the first David Beecroft memorial lecture. There is, in the general confidence of his fitness to carry the world-wide responsibility as head of the Economic Cooperation Administration, a recognition of his major contribution and efforts to bring highway safety into effective unity with our everyday living.

In the ECA program the United States has accepted high international responsibility in the struggle to achieve democracy, which essentially means cooperation and coordination. The principle must be accepted and applied to every major feature of our national life if we are to be an example of our ideology worthy of the emulation of other nations. Highway transport consists of so many elements that we can only secure an excellent standard of service as we approach complete cooperation and coordination. On this premise, it appears to be responsive to the purpose behind the series of papers set in motion by Mr. Beecroft, to explore the present state of the art in the single field of highway design in relation to highway safety.

In this particular and limited field there is urgent need for a major shift in the approach to the determination of the geometric design. Since major changes in our thinking and in our policies applied in this field are indicated by the findings of intensive research now available, it may, by inference, be expected that need for major changes exists in other elements of highway transport. It is hoped that future papers in this series may present the record of the present, and also new concepts in the allied fields of motor-vehicle design, motor-vehicle administration, particularly driver licensing and driver training, traffic laws, enforcement, and safety education designed to fortify driver behavior with disciplined attitudes of mind. These are all interrelated and interdependent, and our mastery of the whole problem of street and highway transport with safety depends upon mastery in each of these fields.

This paper purports only to record the devoted work of many individuals and many organizations to advance the cause of highway safety. The Action Program of the President's Highway Safety Conference, the extensive work of the Automotive Safety Foundation, that of the National Safety Council, and of many others, are reflected in the conclusions reached. The factual data are the product of the Department of Research of the Public Roads Administration under Deputy Commissioner H. S. Fairbank, and in particular result from detailed research of O. K. Normann, Carl Saal, C. W. Prisk, W. P. Walker, A. Taragin, and others who have engaged in the field of traffic research intensively and devotedly over many years.

The application of such data to design practices of Public Roads is the product of the Department of Design under Deputy Commissioner H. E. Hilts and his associates. There is particular response to the advance in design requirements by the Urban Division under Joseph Barnett.

The effort has been made to eliminate personal opinion, and to substitute conclusions supported by detailed observations of controlled research carried on over a sufficient period and with adequate repetition to justify confidence in the results. It is, however, not the intention to reflect an implication of finality. Continuing research will refine and illumine problems which are as yet obscure, but it is certain that driver behavior en masse is the key to safe highway design.

THE BACKGROUND OF HIGHWAY DESIGN

For too long, and in too great degree, highway design has been distorted by the tyranny of wrong concepts. Most important of these in its adverse impacts is the error of thinking of the motor vehicle as static in relation to the highway. In use, the vehicle is dynamic and takes on very different qualities.

When the motor vehicle first appeared, it was held to be a legal interloper on highways whose function was to serve horse-drawn traffic. As the number increased, only minor concessions were made to its higher speed, in the way of superelevated curves—probably borrowed from prior railroad practice. The effects of railway engineering tenets upon highway design are evidenced by the meager tolerances which have been built into our

[1] This paper was presented as the second David Beecroft memorial lecture at a meeting of the Society of Automotive Engineers in Washington, D. C., November 16, 1948.

roads, reflecting operations on fixed tracks not subject to deviation from course. The motor vehicle in motion is highly flexible; but at relatively low densities of traffic the operation begins to take on the character of stream flow, and the individual vehicle loses its identity and its freedom. As an integral part of a traffic stream, each vehicle affects, and is affected by, all of the other vehicles in its own flow line and in contiguous streams.

The old adage—the proper study of mankind is man—is not only scientifically correct, but is the only approach to the problem of highway design with maximum safety. Human behavior at the wheel, with foot on the accelerator rather than the brake, is the all-important criterion for highway design. The implication here is not an invitation to speed, or to design with the objective of high speeds. Driver behavior en masse reflects normally average rural speeds of 42 to 48 miles per hour when reasonable freedom is permitted. As cars are designed, the foot on the accelerator is the normal driving position, and through the accelerator the driver expresses not only his desire to reach an objective, but in addition a wide range of reactions that may be conscious emotions or unconscious reflexes. The driver en masse is not designed for high speeds with safety. Once the mean and range of human behavior are determined by precise measurements extended to thousands of observations until the general pattern emerges, the design limits thus fixed will have sufficient tolerance to provide for the usual departures from the norm. Highway capacity adjusted to traffic volume is a major factor in safe highway design. True economy is served only if this test is met, and thus safety becomes directly a measure of efficient as well as economical design.

The ratio of commercial vehicles to passenger cars has important effects upon the road capacity to carry traffic. In 1904 the ratio of trucks and busses to passenger cars was 1 to 78; by 1910 this ratio had become 1 to 45; in 1920, 1 to 7.3; in 1930, 1 to 6.5; in 1940, 1 to 5.9; and it is expected that for 1948 the ratio will be about 1 to 4.5 (see table 1). The percentage of commercial vehicles in the total daily traffic flow is continuing to increase, and for this reason more adequate recognition of the requirements of such traffic must be incorporated in the design to insure safe, efficient operation of all traffic.

The influence of increasing the percentage of commercial vehicles upon the total capacity of multilane highways, under the favorable conditions of uninterrupted flow, is illustrated in table 2. On two-lane highways this influence on capacity is more severe.

Table 1.—*Registrations of automobiles and of trucks and busses*

Year	Registrations			Ratio of trucks and busses to automobiles
	Automobiles	Trucks and busses	Total	
1904	54, 590	700	55, 290	1 to 78.0
1910	458, 377	10, 123	468, 500	1 to 45.3
1920	8, 131, 522	1, 107, 639	9, 239, 161	1 to 7.3
1930	22, 972, 745	3, 559, 254	26, 531, 999	1 to 6.5
1940	27, 372, 397	4, 663, 027	32, 035, 424	1 to 5.9
1947	30, 718, 852	6, 641, 611	37, 360, 463	1 to 4.6
1948 [1]	33, 225, 000	7, 332, 000	40, 557, 000	1 to 4.5

[1] Estimated.

In the years just prior to 1946 the national fatality rate was about 12.0 deaths per 100 million vehicle-miles of travel. In January 1946, the rate was 12.4. The President's Highway Safety Conference was called in May 1946, and, notwithstanding the bad record of the first 3 months of that year, the upward trend was halted and turned abruptly downward so effectively that the year ended with a rate of 9.8. The wholehearted support, and the devoted interest and efforts of organizations, officials, and the public in putting into effect the Action Program developed through the Conference, are mainly responsible for the reduction in the fatality record to the estimated rate for 1948 of 7.8 per 100 million vehicle-miles. As this rate is lowered we must expect to reach a critical point at which a continuing decrease will prove impossible through education, enforcement, and the other means which have been responsible for the splendid record made since May 1946.

In addition there must be the reduction, through the general use of higher standards of highway design, of the current fantastically high accident potentials. The astronomical number of accidents that do not happen is terrifying. As traffic volume increases, the accident possibilities—that is, the pressures for accidents—build up in geometric ratio. The accident potentials can only be reduced with certainty by reducing the possible conflicts of traffic units.

We know that major reductions in the fatality rate can be made by providing properly designed, modern highway facilities, as is evidenced by the record of highways on which the accident potentials are greatly reduced. The following highways carry large volumes of traffic but have fatality rates as low as one-fifth to one-third of the national average because of the controlled-access design in which conflicts of all kinds—cross traffic, pedestrians, and traffic entering along the roadsides—have been materially reduced or eliminated:

	Death rate per 100 million vehicle-miles
Merritt and Wilbur Cross Parkways (1946)	2. 5
Chicago Outer Drive (1946)	2. 9
Riverside Drive, California (1941–44)	3. 0
Arroyo Seco Parkway (1941–44)	3. 9
Metropolitan New York Parkway System (1938–48)	2. 5
Pentagon network, Washington, D. C. (1942–48)	1. 5

The Pentagon network is composed of 17 miles of one-way through roads, 7.7 miles of one-way connecting ramps, and 2.3 miles of two-way local service roads—a total of 27 miles—on which travel for the period of slightly more than 6 years since its construction is estimated at 337,500,000 vehicle-miles. Five persons, three of whom were pedestrians, have been killed during this period. Further reference will be made to the observations of driver behavior on the roads of this system.

Under the inspired, at times militant, leadership of Mr. Robert Moses, the Metropolitan New York Parkway System has set an example of highways designed on a scale commensurate with traffic requirements of reasonable speed with safety. The system, steadily expanding, now consists of 164 miles of multilane roadway, much of which is divided. The obstacles which have been met are somewhat indicated by the required 208 bridges. There are two details of design, which cannot be over-commended, that have been or are being put into practice as safety measures. One is the installation of center curbs on the earlier undivided parkways: these have successfully eliminated the deadly head-on collisions. The other is the installation of paving blocks to support heavy vehicles on the shoulders when disabled.

The Action Program of the President's Highway Safety Conference lays great and proper stress upon the acceptance of individual responsibility for safety on the highways. Educators are increasingly stressing the importance of "true personal participation" in contrast to activity unrelated to the subject of study. Contrariness and individualism in respect to a desirable course of conduct disappear once the individual accepts his responsibility for safety of himself and others on the highway.

The responsibility of the individual toward the public to preserve highway safety should be activated by school instruction, town meetings, community forums, local coordinating and action committees, support by the press, discussion groups—all of which are calculated to impress the individual with his responsibility to the public and in this way mold or remold the habits and attitudes relating to safety on the highways.

When these approaches to the problem of accidents have been used, there still remains the very pertinent query, "What is the proper attitude of the public toward the driver?" Basically, the individual driver has his limitations of habits, of capabilities, and the handicaps of his own perception and reaction times. Each driver in a stream of traffic loses his freedom of action in a high degree, and is circumscribed by the actions of others. His performance is limited by, and becomes part of, the traffic flow, which is a composite of the reactions of a multitude of drivers. Thus the responsibility of the public to the individual driver is, first, to determine the characteristics of traffic flow lines, and second, in harmony with this knowledge, to provide the facilities

Table 2.—*Effect of commercial vehicles on practical capacities of multilane highways*

Proportion of commercial vehicles to total traffic	Percentage of passenger-car capacity on flat terrain	
	Flat terrain	Rolling terrain
Percent	*Percent*	*Percent*
None	100	100
10	91	77
20	83	63

for safe use. The dynamics of these speeding streams of traffic most directly influence the design features which we term geometric. These are alinement, profile, plan of intersection, clearances, horizontal dimensions of the highway cross section, and the many details which we group under the general term of highway design.

PRINCIPAL HAZARDS OF THE DESIGN CHARACTERISTICS OF EXISTING HIGHWAYS

The outstanding hazard of our streets and highways is undercapacity for the traffic load. Nearly half of the rural highways carrying 1,000 vehicles per day and over are less than 20 feet in width. On each mile of such highway, over 60 times per hour, or once each minute, a vehicle is encroaching upon the left lane when meeting an oncoming vehicle. Expanding these figures to the many miles of rural highways in this country, the accident potential reaches unrealized dimensions. The length of time that a vehicle in passing another vehicle occupies the lane used by the oncoming traffic, depends upon the width of pavement. On an 18-foot pavement vehicles occupying the left lane require 43 percent longer in passing than on pavements 24 feet wide. The pavement of the lesser width is not only more hazardous, but provides only 70 percent of the capacity of the 24-foot pavement.

Adequate shoulders are necessary to the efficiency of motor-vehicle operation. They are effective in increasing the traffic capacity of the highway, since bituminous-treated shoulders 4 feet or more in width adjacent to 18- and 20-foot pavements, as compared with grass or gravel shoulders, increase the effective surface width approximately 2 feet. Increasing the effective pavement width results in fewer vehicles encroaching upon the left lane of traffic, and thus increases the traffic-carrying capacity of the highway.

Shoulders also play a very important role in accident prevention in both rural and suburban areas where pedestrians are involved. In 1947, 25 percent of all rural pedestrian fatalities occurred while pedestrians were walking on the roadway. Were adequate shoulders available of the character necessary to provide usable footpaths, many lives would be saved.

To gain the advantage of adequate shoulders, there should be no vertical obstructions such as retaining walls, bridge trusses, utility poles, guard rails, parked vehicles, or other objects near the traveled way. Obstacles at the edge of a 10-foot lane cause vehicles to travel 2.6 feet farther from the edge of the pavement than when the obstacles are not present. Even for lanes 12 feet wide the same positioning of obstacles causes vehicles to travel 1.8 feet farther from the pavement edge. Obstacles 4 feet or more from the pavement edge have only minor effect. The roadside obstacles, therefore, have the effect of reducing the pavement width.

Adequate width of shoulder and a suitable surface for parked vehicles are necessary for every highway regardless of its width. On the Merritt Parkway in Connecticut, two-thirds of the fatalities resulting from collisions between vehicles have involved a parked vehicle on the roadway. The German Autobahnen were built with divided paved roadways, each 29.25 feet in width, thus providing capacity greatly in excess of normal requirements. The shoulder width was 1 meter. In 1938, after a very few years of use of a limited mileage, Inspector-General Dr. Fritz Todt, the official in charge, stated that the decision had been made to add wider shoulders because of numerous accidents caused by vehicles standing on the pavements. This experience is compelling in establishing proof of the danger inherent in the vehicle parked on the roadway. Every other element which we rate as contributory was absent, and the standing vehicle per se was the cause of the accidents.

Where sight distances on two-lane roads are so short that passing would be hazardous, it is customary to stripe no-passing zones. Observations were made on a 3,400-foot length of highway, of which about one-fourth was marked "no passing" by reason of sight distances below 400 feet. Of the total number of passings made within this 3,400-foot length, more than 10 percent were started or completed in the restricted zone. On roads where no-passing restrictions comprise as much as 40 percent of their length, the volume of traffic that can be satisfactorily and safely accommodated is only about 80 percent of that which can be carried on a similar highway free from sight distance restrictions.

The road margin is the critical line of hazard on many of our existing highways. Recognition of this situation appears in a project just initiated in Michigan, in which a study of the locale of accidents with respect to taverns, gas stations, restaurants, and other roadside establishments is to supplement the more conventional facts analyzed in studies of traffic accidents. The unrestricted use of the road margin for entrance and exit at such locations causes driver confusion, disorderly parking, and other operating hazards. The growth in the number and popularity of outdoor drive-in theaters is but a single example of the commercial exploitation of the roadside. The safety of motoring audiences, as they enter or leave these establishments in large numbers, has become a necessary concern of highway authorities because of the effect on operating conditions on the adjoining highways. The most difficult obstacles the highway engineer faces in his effort to build safety into the highways are the lack of legal authority and effective legal machinery provided to acquire sufficient right-of-way and to control the entrance to the public highway from abutting property.

A characteristic serious fault of drivers is that of following too closely a car traveling in the same lane. The speed and the interval between the cars do not permit adequate perception and reaction time for the rear car to stop if the car ahead is brought to an unexpected halt. This type of accident has the potentials of a whole series of accidents, of tying up traffic, of causing much property damage, and sometimes serious or fatal accidents. The grave implications of this very common practice are indicated in table 3. The perception and reaction times vary with individuals, but a safe assumption would hardly be less than 1.5 seconds. Tests of drivers have shown that the minimum reasonable reaction time alone is three-fourths of a second.

The average time spacing between vehicles following at the same speed, for all speeds and for both two- and four-lane roads, is 1.58 seconds; 16.2 percent of the traffic will have a time spacing of 0.75 seconds or about one-half the average.

Table 3.—*Average spacing between vehicles following other vehicles closely in the same lane at the same speed* [1]

Speed	Distance of travel per second	Two-lane highways		Four-lane divided highways			
				Left lane		Right lane	
		Distance spacing	Time spacing	Distance spacing	Time spacing	Distance spacing	Time spacing
M. p. h.	*Ft./sec.*	*Feet*	*Seconds*	*Feet*	*Seconds*	*Feet*	*Seconds*
10	14.7	30	2.04				
20	29.3	42	1.43				
30	44.0	62	1.41	60	1.36	80	1.82
40	58.7	90	1.53	77	1.31	105	1.79
50	73.4	122	1.66	107	1.46	130	1.77
60	88.0	162	1.84	139	1.58	156	1.77

[1] Spacing measured from the rear bumper of one car to the front bumper of the following car.

THE COMPONENTS OF ROAD DESIGN FOR SAFE OPERATION, AS INDICATED BY DRIVER BEHAVIOR

Alinement

The alinement must be adjusted to the volume and type of traffic, the driving characteristics of the operators, and the dynamic effects of the mass movement. The alinement selected, whether the highway be two, four, or more lanes wide, determines how effectively and safely the completed facility will meet the demands of traffic. Sight distances between 1,500 and 2,000 feet long are essential on any rural two-lane highway. The percentage of the length where sight distances of this magnitude should be available will depend on the volume and type of traffic which the highway must accommodate. On our main highways carrying high-speed through traffic, a sight distance of at least 1,500 feet should be available on a

The need for wide shoulders is evident here.

minimum of 60 percent of the route length if the highway is to accommodate peak traffic volumes as high as 500 vehicles per hour with safety. If it is unavoidable to introduce curvature which restricts the sight distance, it is essential to select an alinement free from sudden changes that come as a surprise to the operator. Curves at the ends of long tangents are definitely more hazardous than the same curves located where the general alinement is made up of a series of curves.

Practical Working Capacities

Accidents are inevitable on overloaded highways. Safe highways must have sufficient capacity to permit drivers to operate at reasonable speeds. On main rural highways drivers generally accept, as reasonable, average running speeds of 45 to 50 miles per hour during the peak volumes. On urban expressways, a speed of 30 to 35 miles per hour during peak traffic volumes is reasonable.

The working capacities for modern rural roads in terms of passenger cars per lane are 450 per hour for a two-lane road, and 1,000 per hour for the lanes in the direction of heavier travel on multilane roads. At these traffic volumes, under ideal roadway conditions, the drivers who so desire can safely travel at a speed of 45 to 50 miles per hour on rural roads, although the average speed of all vehicles at any given point will be about 42 miles per hour as compared to an average speed of about 48 miles per hour during low traffic densities.

In urban areas the maximum practical working capacity for a modern multilane expressway is 1,500 passenger cars per hour for each lane in the direction of heavier travel. At this volume, drivers who so desire can safely average 35 to 40 miles per hour, and the average speed of all vehicles will be between 30 and 35 miles per hour. At this working point, unusually high volumes that occur frequently for short periods can be handled without complete congestion.

Lane Widths

The lane widths used for the design of our highways must be based on vehicles in motion; not on the actual size of the vehicles standing still. Most of our main highways have sufficient traffic of commercial vehicles to require 12-foot lanes for safe operating conditions. Eleven-foot lanes must only be considered for highways carrying less than 1,000 vehicles per day. The reduction in capacity by narrowing lanes is not only unsafe; it is definitely uneconomical—the cost of the additional width is less than the proportional increase in capacity. The effect of lane width on capacity is shown in table 4.

Two-Lane Highway Limitations

The accident experience on two-lane highways shows that the death rate per million vehicle-miles increases with an increase in the traffic volume. Operating conditions on the average two-lane highway are not satisfactory when traffic volumes exceed 4,000 vehicles per day in flat terrain, or 3,300 vehicles per day in rolling terrain. It is only for extremely low daily traffic volumes that a two-lane highway can safely accommodate a vehicle traveling over 60 miles per hour, regardless of excellent alinement. Design speeds that exceed 60 miles per hour, to provide safe operating conditions, require four-lane divided highways if the volume exceeds 3,000 vehicles per day. On free-flowing highways where the higher speeds are safer, this very fact automatically lowers the top range by permitting a constant rate which satisfies most drivers. The average actual rate for two-lane highways of modern superior design is about 48 miles per hour.

There is confusion of the terms "overload" and "congestion." Congestion approaching stagnation may lower the frequency of serious accidents, but at the same time it defeats the utility of highway transport. Any concept that congested highways are safer completely overlooks the fact that each route has its own characteristic pattern of daily use. It may have an hour or a fraction of an hour of congestion once or twice daily, at peak traffic periods. These periods of congestion causing stagnation or incipient stagnation may be productive of numerous but not fatal accidents. The remainder of each 24 hours constitutes 90 to 95 percent of the time, during which there are much longer periods when the route is overloaded and a breeder of accidents.

Multilane Highways

Multilane highways of the divided type will accommodate three to six times as many vehicles per lane as a two-lane highway, and provide greater safety. For average conditions, the width should be increased from four to six lanes when the volume exceeds 18,000 vehicles per day in rural areas, or 32,000 vehicles per day in urban areas. In certain areas of the country where seasonal, daily, and hourly fluctuations in traffic flow vary from the average condition, the practical capacity of a four-lane divided highway might be as much as 30 percent higher or lower than the above volumes.

Shoulders are Essential

Shoulders that will accommodate disabled vehicles and that may be used by moving vehicles in cases of emergency, during any weather conditions, are essential for safety on main highways. Adequate shoulders are also essential to realize the full capacity of the surface width. Without adequate shoulders, one disabled vehicle can reduce the capacity of both two-lane and multilane highways during peak periods by as much as 60 percent. There is at least one disabled vehicle for every 20,000 vehicle-miles of travel on our main highways.[2] Furthermore, adequate shoulders increase the effective surface width for traffic when no disabled vehicle is present. Without a place of refuge outside the traffic lanes, one disabled vehicle can reduce the capacity of a highway by more than one lane, especially if the lanes are less than 12 feet wide.

The disabled vehicle blocks one lane and reduces the capacity of adjoining lanes by restricting vehicle speeds. It may block all lanes in one direction, completely stopping traffic in that direction. The maximum capacity of a traffic lane with vehicles moving at 20 miles per hour is only 87 percent of its capacity at 30 miles per hour; at 10 miles per hour, only about 50 percent. A minor accident which causes a reduction in speed can result in complete congestion when a facility is operating near its capacity.

Table 4.—*Effect of lane width on capacity*

Lane width	Percentage of 12-foot lane capacity		
	Two-lane rural roads		Multilane expressways at practical urban capacities
	At possible capacities	At practical capacities	
Feet	*Percent*	*Percent*	*Percent*
12	100	100	100
11	88	86	97
10	81	77	91
9	76	70	81

[2] On the Oakland Bay Bridge, for example, during the peak hour when 6,700 vehicles use the 6-mile structure, an average of two vehicle break-downs can be expected, which will seriously affect the traffic capacity of the bridge.

Curbs and Lateral Clearances

Any vertical member adjacent to a roadway constitutes a safety hazard and is an obstruction to the free movement of traffic, unless it is 6 feet removed from the pavement edge. High vertical curbs of the so-called nonmountable type fall within this category, and when required they should be 6 feet removed from the traveled way in order to have no effect on traffic. If they are 3 or 4 feet away, however, their effect will not be critical. A low sloping curb may be used adjacent to the roadway surface in conjunction with a high curb further removed.

Drainage

The provision of side ditches, gutters, and drainage structures, sufficient to prevent mud and debris from washing onto or collecting on the roadway surface during storms, is a necessary safety measure. Such foreign materials constitute hazards in themselves by creating slippery pavement conditions; but of greater importance is the hazard created by drivers who, in attempting to avoid such materials, bring their vehicles into the paths of other vehicles. In the colder regions the formation of ice on pavements as a result of water collecting in poorly drained places presents a serious safety problem.

Highway Intersections

To provide safe operation at intersections, on rural through roads carrying an average traffic of 5,000 vehicles per day and over, the grades should preferably be separated at intersections with other major roads.

Chief among the reasons for safety violations at intersections is congestion. The driver whose patience is exhausted is here the dangerous operator. Aside from relief of congestion, there are a number of measures that aid in minimizing hazards at intersections. Among these measures the following rank high:

1. Corner sight distances consistent with the operating speeds of traffic, with due consideration of the method of traffic control employed.

2. Safety islands for pedestrians at busy intersections where streets are wide and pedestrian traffic is heavy.

3. Separated turning lanes, especially on high-speed or moderately high-speed divided highways, to enable turning vehicles to decelerate and stop, if necessary, clear of the lanes used by through traffic.

Railroad Grade Crossings

All main-line railroad grade crossings of highways which carry substantial traffic flows should be separated. Because of the cost and the pressures for other highway improvements, the elimination of crossing hazards by construction of separation structures will be delayed at many locations. Consequently, grade-crossing protection should proceed rapidly. Particularly effective are the installations of the short-arm automatic gates. These have some obvious maintenance and cost advantages over the long-arm gate installations,

An accident a mile away resulted in this concentration of traffic.

and provide a barrier which experience has proven adequate. Though the combined short-arm gate and flashing signal installation is somewhat more expensive than that of the flashing signal alone, this type should be uniformly used for all main-line crossings at grade, regardless of the number of tracks. This position is reflected in the present policies of the Public Roads Administration.

Loading Recesses

The destined function of practically all of our highway mileage, both new and old, is to provide service to all classes of traffic. The peculiar needs of commercial and mass-transit vehicles, not only in their steady flow movements but also at their terminals, transfer points, and roadside stops, require attention in shaping the physical features of the roadway. The frequent stopping of transit or commercial vehicles in the moving traffic stream is a serious traffic hazard. Wherever possible, and in all instances on new highways designed as routes for mass transit, proper facilities should be provided off the through lanes for transit-vehicle passenger loading and unloading. These roadways should be designed to expedite movements of the transit vehicle as well as those of the more numerous private passenger car.

Through the provision of more attractive and convenient mass transportation, there is the possibility of shrinking our urban traffic problems to more moderate limits. Efficiently designed off-street loading bays and terminals for trucks serve a similar purpose, particularly in the central core of our cities where street space is wholly inadequate for the moving traffic.

Uniform Signs and Signals

Traffic-control devices, though relatively inexpensive accessories to street and highway design, are significant aids to safe highway operation. Well designed and located highway signs, signals, and markings eliminate or relieve many of the elements of surprise that characterize certain combinations of design and traffic-operation features. The planning of new highway facilities should always be accompanied by thought as to the traffic control, to hold restrictive measures to a minimum. Such preliminary considerations also offer certain correlative benefits in the joint planning of design and control elements on the new facility. The control will thus be tailored into the design at the most appropriate stage, and not left for piecemeal application after completion of the construction. The Manual on Uniform Traffic Control Devices should be rigorously followed. The manual has become the single legal standard for use on all Federal-aid projects.

Speed Control

Reasonable speed is a necessary element of efficient highway transportation, yet speed is so frequently pointed to as a traffic accident cause that the dictum has wide acceptance that traffic accidents and higher speeds go hand in hand, and that there can be no one facility that possesses the characteristics of speed and safety. Numerous attempts to relate speed and accidents have had but meager success. Speed definitely increases the severity of accidents, and it has been demonstrated from studies in New York, New Jersey, and Connecticut that fast drivers are guilty of more traffic violations and are involved in more accidents than those who operate at more moderate speeds. But we also find that our modern highways with excellent safety records are accommodating traffic speeds definitely above those found on routes of inferior design.

The answer is certain. The newer facilities have been purposely planned to meet the higher speed demand, and are distinguished by uniformity in those design features that relate to safe accommodation of faster-moving traffic.

Safe vehicle speeds thus depend in large measure on characteristics of the roadway. With approximately 400 billion vehicle-miles of travel annually, it can hardly be expected that police activity on speed-law enforcement will be effective for more than a small fraction of this total. Speed controls, where applied, must be adapted to the circumstances. On our highway systems, both urban and rural, there is a wide range of ability to accommodate speed. This is true not only for the nation but for any State, or for any appreciable length of highway within a given State.

Thus, the establishment of a uniform legal speed limit for rural and urban areas is not a solution. Such a policy is unsound. A limit set at the maximum safe speed for the poorest section of highway is obviously an invitation to disregard the law. As shown in table 5, 37 of the States now exercise the authority to post speed limits differing from the speed that has been set as a general maximum. The

Table 5.—*State-wide speed limits and speed zoning authority*

Rural State-wide speed limit	Number of States	States with rural speed zoning authority
No limit	14	10
60 m. p. h.	8	8
55 m. p. h.	7	7
50 m. p. h.	10	7
45 m. p. h.	4	3
40 m. p. h.	2	0
35 m. p. h.	2	2
25 m. p. h.	1	0
	48	37

Table 6.—*Suggested zone speeds for safe operation on curves*

Degree of curvature	Radius of curvature	Suggested zone speed[1]
	Feet	*M. p. h.*
3	1,910	60
4	1,433	53
5	1,146	48
6	955	43
7	819	40
8	717	37
9	637	35
10	574	33
11	522	32
12	478	31
13	442	30
14	410	29

[1] Before zoning speeds are posted the curves should be tested by competent observers to determine the safe speed for the particular conditions.

rural State-wide daytime limits range from 25 miles per hour in 1 State to no limit in 14 States. Of the 29 States having general limits above 50 miles per hour, all but four are using the device of zoning to encourage judicious use of the highway with respect to speed. Much more needs to be known about the principles of successful zoning, but sufficient study has been given to determine that the signing must be realistic and possess considerable self-enforcing value. The establishment of frequent, unwarrantedly low limits breaks respect for the entire zoning device, which is reflected in abuses of other essential traffic regulatory measures. Suggested zone speeds for safe operation on curves are shown in table 6.

Efficient Operating Grades for Trucks

Table 7 shows the distance that light-powered trucks with gross weights of 30,000 pounds, and medium-powered trucks with gross weights of 40,000 pounds, can go up various grades before their speeds are lowered to 30 miles per hour. Medium-powered trucks with gross weights of 40,000 pounds will probably not occur with sufficient frequency on most of our main rural highways to justify basing the road design on these vehicles alone. Light-powered trucks with gross weights of 30,000 pounds, however, will occur with sufficient frequency on most main highways to affect seriously the operation of passenger vehicles on the highway unless they are considered when selecting the alinement and other design features of the road. It will be noted that the length of grade that reduces the speed to 30 miles per hour is approximately the same for the light-powered trucks with the gross weights of 30,000 pounds as for the medium-powered trucks with gross weights of 40,000 pounds.

A third lane on the uphill side will prevent the slow-moving trucks from unduly interfering with other traffic if the third lane is started at the proper distance from the bottom of the grade, as shown in table 7. In certain types of terrain, however, especially where the road is located on the side of a steep hill or mountain, it may be impractical to provide an additional lane. Since the problem must be met on main routes, there are the alternatives of improved alinement for routes of light travel, or the addition of a new one-way, two-lane roadway on a separate location. In the latter case the old plus the new roadway would provide two lanes in each direction.

Far too little attention is given the fact that farm production is moving to the markets and to the railroads in trucks. In a constantly increasing degree the secondary or farm-to-market roads must be designed for truck operation; and efficient, safe operation requires low gradients. The lowest unit-cost item in road construction today is earth excavation, and there is no reasonable excuse to neglect the economy that will accrue to the farms by requiring additional excavation in designing farm-to-market roads, to provide low grades and good alinement.

From the angle of safety only, the most serious result of grades is the restricted sight distance at the crest, or of the erroneous design policy, quite generally practiced, of introducing short sight distance curves to lower the percentage of gradient.

The Less-than-Traffic-Speed Driver

On week ends and holidays the most fervent anathemas of other drivers are directed toward the driver who fails to maintain the average traffic speed. This attitude mistakes the effect for the cause. The collection of a queue of cars with frustrated drivers behind a car moving slower than the average traffic speed is a sure indication of an overloaded highway. The same conditions apply as in the case of the slow-moving truck. The fault is the highway, not the slow driver. Too narrow traffic lanes and too restricted sight distances are the most common defects. The slow driver is not likely to disappear from the roads; the remedy lies in eliminating his accident potential by improved road design.

Capacities of Express Highways

At the present time our main highways carry about 20 percent commercial traffic. During peak hours, however, commercial vehicles are generally in the neighborhood of 10 percent of the total traffic. In rural areas, two-thirds of the traffic is generally in one direction during the peak hours. In urban areas, the distribution by direction will generally vary from two-thirds in one direction in the outlying areas to an equal distribution each way in the central business districts. It is generally not feasible to obtain a perfect alinement having no sight distance restrictions, on two-lane highways. Even in flat terrain it is difficult to obtain more than 80 or 90 percent of the road with a sight distance over 1,500 feet, and in rolling terrain it is difficult to obtain more than 60 percent of the road with a sight distance over 1,500 feet. The total hourly capacities for two- and four-lane highways in rural and urban areas, for these average conditions, are as shown in table 8.

Table 7.—*Distance from bottom of grade at which truck speed would be reduced to 30 miles per hour*[1]

Grade	Light-powered trucks with gross weights of 30,000 pounds		Medium-powered trucks with gross weights of 40,000 pounds	
	Distance from bottom of grade	Vertical climb	Distance from bottom of grade	Vertical climb
Percent	*Feet*	*Feet*	*Feet*	*Feet*
2	2,000	40	1,780	36
3	1,090	33	1,035	31
4	760	30	740	30
5	570	29	550	28
6	470	28	450	27
7	400	28	390	27
8	325	26	320	26

[1] Assuming an approach speed of 40 miles per hour. Bad alinement, weak and narrow bridges, or other hazardous conditions at the bottom of the grade would make these speeds unsafe.

Table 8.—*Hourly and annual average 24-hour design capacities for express highways*[1]

Type of area and terrain	Hourly design capacities		Annual average 24-hour design capacities[2]	
	2-lane highway	4-lane highway	2-laue highway	4-lane highway
Rural areas:[3]				
Flat terrain[4]	600	2,800	4,000	18,700
Rolling terrain[5]	500	2,500	3,300	16,700
Urban areas:[6]				
Flat terrain[4]	---	4,550	---	38,000
Rolling terrain[5]	---	3,850	---	32,000

[1] Based on 10 percent commercial vehicles during peak hours.
[2] Based on a 30th highest hour of 15 percent of the annual average 24-hour traffic for rural areas, and a 30th highest hour of 12 perceut of the annual average 24-hour traffic for urban areas.
[3] Based on two-thirds of the traffic in one direction and an operating speed of 50 m. p. h.
[4] Based on sight distance of at least 1,500 feet on 80 percent of the 2-lane highways.
[5] Based on sight distance of at least 1,500 feet on 60 percent of the 2-laue highways.
[6] Based on 60 percent of the traffic iu one direction and an operating speed of 35–40 m. p. h.

When relating the hourly design capacities to annual traffic volumes, fluctuation in traffic flow must be considered. Comprehensive studies of the fluctuation in traffic flow on rural highways show that the traffic density somewhere between the thirtieth to the fiftieth highest hourly traffic volumes of the year should be selected for design purposes. On the average rural highway, the thirtieth highest hour of the year is about 15 percent of the annual average 24-hour traffic volume. In urban areas it averages about 12 percent of the annual average traffic volume. Table 8 shows the annual average 24-hour traffic volumes that can be used for design purposes at locations where there is the average fluctuation in traffic flow. Values as much as 30 percent higher or lower than these may be applicable to a specific location, depending on the seasonal, daily, and hourly fluctuation, and the distribution of peak-hour traffic by directions.

A four-lane expressway of modern design, with controlled access, will accommodate as much traffic, at approximately twice the average speed, as—

1. Five ordinary city streets, each 40 feet in width, with parking prohibited.
2. Eight ordinary city streets, each 42 feet wide, with parking on both sides.
3. Five ordinary city streets, each 68 feet wide, with parking on both sides.

4. About three ordinary city streets, each 68 feet wide, with parking prohibited.

By "ordinary city streets" is meant those that have the average amount of left-turning movements and pedestrian interference prevalent in downtown areas.

Brakes

The hazardous conditions created by too short spacing between cars following in the same lane are emphasized by the studies of brake performance. Safe vehicle performance depends to a greater degree upon adequate, well-maintained brakes than upon any other single element of vehicle design. Likewise, the brakes betray most surely the failures and neglect of the driver or owner. Studies by the Public Roads Administration just before the war disclosed that over 40 percent of the passenger cars tested on the highway had zero pedal reserve, i. e., the pedal was flat on the floor board after an emergency stop from 20 miles per hour.

Brakes are a common alibi in many accidents; sometimes the true cause, but more often not. If the brakes fail, the responsibility lies usually with the driver. He has not maintained them properly.

The widely distributed studies of brake performance show that the condition existing in 1942 was not a rosy one. It is probably worse today. Only 62 percent of the passenger cars, 18 percent of the two-axle trucks, and 3 percent of the three-axle combination units could stop in 30 feet or less from a speed of 20 miles per hour.

There is abundant confirmation that the braking systems of passenger cars and especially of commercial vehicles are too commonly poorly maintained or grossly inadequate. An analysis of the results indicates that this level of performance can be greatly improved. Passenger cars and two-axle trucks can be made to stop from 20 miles an hour in 30 feet or less. The combination units now on the road may not be able to do that well, considering the lag that exists from the time that the pedal is touched until the brake shoes actually contact the drums. However, it is certain that with proper maintenance of well-designed braking systems, a much better showing can be expected. It is heartening, in this respect, to read that the Pennsylvania Motor Truck Association's Engineering Subcommittee has been able to stop a 62,000-pound, two-axle tractor and tandem-axle semitrailer in an average distance of 27¾ feet, from 20 miles per hour. There is every indication that the automotive engineers who cooperated in the tests have succeeded in reducing to a marked degree the inherent brake lag.

The tests of vehicles as they are now operating on the roads clearly show that the proper maintenance of brakes is not as yet being generally accomplished, and that a more stringent enforcement policy is necessary.

The importance of brakes in relation to driver behavior is emphasized by the data in table 3, showing distance spacing and time interval between cars traveling in the same lane. If the driver behind has instant perception time (which is impossible) and the average reaction time of three-fourths of a second, and if his speed is only 30 miles per hour, he has two-thirds of a second for his brake to stop his car if the car ahead stops suddenly. Under such typical circumstances, when an accident occurs do the driver's brakes fail to work or does the driver fail to work the brakes? The answer is one major reason why insurance rates are going up.

THE PARADOX OF DRIVER BEHAVIOR

"We know so much about roads that is not so," has long handicapped correct thinking. We do know now much of driver behavior that without positive proof we think could not be true. For example, the relationships of accident frequency and accident potentials with the details of highway design have been described. The accident rate on two-lane rural highways carrying less than 1,000 vehicles per day is approximately half that on similar highways where the volume is in the range of 8,000 or more vehicles per day. That is, on overloaded highways or underdesigned highways the accident rate increases with traffic volume. On such highways, we would logically think drivers would become more careful. Paradoxically, driver behavior becomes worse as the road becomes more inadequate to carry safely the traffic volume. Under conditions where the driver should become more responsible he becomes less so. Contrariwise, evidence supports the conclusion that as the standard of design is raised, driver performance becomes more responsible. This observation of driver behavior undoubtedly reflects the ability of the average mind and nerves to function more safely if tension beyond necessary alertness is relieved and apprehension is absent.

Here is the record of traffic operation that has emerged from the studies on the Pentagon network of roads over the past 6 years. The system was designed and built by Public Roads to serve an estimated population of 50,000 office workers in the new defense establishments, all of which were transferred almost overnight across the Potomac River to a previously unoccupied area in Virginia. The problem was to move the major part of this population in and out daily by motor vehicle over the existing three Potomac River crossings—Key Bridge, Memorial Bridge, and Highway Bridge.

Excluding the traffic over Memorial Bridge which uses Lee Boulevard, and other traffic which does not actually traverse a portion of the 17 miles of through routes, it is estimated that there are now 85,000 trips per day on the Pentagon system. Including the traffic on roads immediately tributary to the system, there are about 110,000 trips on these roadways daily. This is a conservative figure, since the three bridges which the system serves are now carrying an average traffic of more than 139,000 vehicles per day and a peak-month traffic of 149,500 vehicles per day. The system now produces about 67 million vehicle-miles of travel per year. In the 6-year period, five fatalities have resulted from this operation, three of whom were pedestrians. Only one fatality was caused by a vehicle traveling at a relatively high rate of speed, leaving the road and overturning. For this 6-year period the accident rate was 1.5 fatalities per 100 million vehicle-miles. In this experience lies the assurance of what can be accomplished through safe road design, but the record of driver behavior is the astonishing feature.

During the war there was a speed limit of 35 miles per hour on both the Pentagon network and on the Shirley Highway which extends southward from it. The 35 m. p. h. speed signs were not removed until several months after the war. Prior to their removal, speed studies were conducted at several points on the network and on the Shirley Highway. These studies were repeated after changing the 35 m. p. h. signs to speed limit signs of 40 m. p. h. on the network and 50 m. p. h. on the Shirley Highway. Changing the speed limit on the network from 35 to 40 m. p. h. had no effect on the average speed, but reduced the number of vehicles traveling over 50 m. p. h. by 11 percent and those going over 40 m. p. h. by 20 percent. Changing the signs on the Shirley Highway from 35 to 50 m. p. h. reduced the number of vehicles traveling over 50 m. p. h. by 32 percent.

On the through routes of the Pentagon network, where there are no traffic police, no traffic-control lights, and no enforcement of the 40 m. p. h. posted speed limit, the average speed is 37.6 m. p. h., and 12 percent of the vehicles exceed 45 m. p. h. On nearby Lee Boulevard, where safe speeds are much lower and the posted speed limit of 35 m. p. h. is enforced by frequent patrolling by traffic police, the average speed during light traffic volumes is 37.8 m. p. h., with 11 percent of the vehicles exceeding 45 m. p. h. and 36 percent exceeding 40 m. p. h. Thus on the network, where drivers can legally maintain reasonable speeds at all times, they travel no faster without enforcement than they do on Lee Boulevard, where safe speeds are not as high and the normal enforcement is present. Furthermore, three times as high a percentage of the drivers exceed a 5 m. p. h. tolerance of the posted limit on Lee Boulevard as on the Pentagon network.

The present traffic volume on the network is 800 vehicles per hour per lane at locations where three lanes are available for movement in one direction. At the principal mixing lanes, a flow of 1,150 vehicles per lane occurs where four lanes are available for one-direction movement. The principal mixing lanes are now loaded to their possible capacity at certain peak periods. Other points on the network are loaded to approximately two-thirds of their practical safe capacities. All three of the Potomac River bridges are loaded beyond their practical, safe capacities, and at times the demand actually exceeds the possible capacity of the bridges with their present approaches. The condition will be relieved materially by the completion of the two four-lane, one-way bridges now under construction.

DESIRE LINES OF TRAVEL

In harmony with the premise that roads must be designed on a basis of driver behavior is the relatively new technique of planning the system of arterial roads to serve the heavy flow lines of metropolitan areas on the basis of origin and destination surveys. These surveys disclose the desire lines of major travel. The pattern which emerges is that of a wheel, usually somewhat distorted in shape, but essentially providing radial lines from the urban center which are joined by concentric circles at spaced distances from the center. The concept has the two major objectives of providing free-flowing arterial routes in and out of the center, and of separating through, longer-distance traffic from local vehicle and pedestrian interferences. Also, such a plan will keep a high percentage of the total traffic out of the more congested center of the city. Driver behavior can be relied upon to exercise a high selectivity of route if the facilities provide a choice. Under such circumstances the driver will choose the route that will give him the most direct access to his destination, but at the same time will give him the shortest distance of travel through the more congested downtown streets.

The Pentagon system serves three bridges across the Potomac River, leading into the downtown streets of Washington. These bridges are interconnected by the system, and the driver from the Virginia suburban districts had, until the bridges became so congested as they are today, a choice among the bridges of that route which would require him to travel the least distance on, or would keep him off, the congested downtown city streets. The degree to which he accomplished this purpose without any imposed controls is amazing, as is shown by the data in table 9. The actual distribution is representative of the free choice of traffic between origins and destinations during a period when the traffic volumes were relatively low and there was not sufficient congestion on the bridges to alter the desired usage.

A study of the origins and destinations of traffic using the three bridges indicates that the slight percentage differences between the shortest routes and the actual routes used probably resulted from traffic routing itself, in its use of the three bridges, to the bridge that would permit the shortest travel distance on the Washington central city streets.

Table 9.—*Distribution of traffic on the Potomac River bridges*

Bridge	Actual distribution		Calculated shortest-route distribution [1]
	Aug. 16, 1944	Sept. 1, 1943 to Aug. 31, 1944	
	Percent	*Percent*	*Percent*
Highway Bridge	41.8	42.6	40.2
Memorial Bridge	33.0	31.5	32.7
Key Bridge	25.2	25.9	27.1
Total	100.0	100.0	100.0

[1] Assumed usage of shortest route between origin and destination.

HIGHWAY SYSTEM PLANNING

Finally, this discussion is directed to the essentials of today's program of highway improvement. Since 1934—nearly one and one-half decades—the State-wide highway planning surveys have been accumulating the facts of the amount and characteristics of the traffic use of our highways. State by State, undoubtedly more is known about the highways, their use, and their needs, than about any other major public undertaking. Until recently this wealth of material had not been generally analyzed and organized to serve its most important purpose as conceived when the surveys were started. In 1945 the California State Legislature, recognizing the need for a long-range highway improvement plan, established a study committee to organize the available material and to secure new data necessary to bring together a complete engineering and economic report. The completion of the report, the legislative recommendations, the legislative debate, and the adoption of the major recommendations, are now a part of the State's highway history. California has a well-rounded, long-term, modern highway improvement program. With this successful guide, many other States have now in active operation legislative study committees, which, with the competent technical assistance of the Automotive Safety Foundation, are maturing long-range programs of comparable caliber. The planning is designed to serve both economic needs and safety of use. Since the future reduction in highway fatality toll, as has been shown, will depend so largely upon safer roads, no effort by the Foundation can result in a greater contribution to highway safety.

There are also implications of vehicle design that can be well heeded. For example, it is certain the driver needs to have a better sight of the right-hand edge of the road. Protective bumpers, front and rear, capable of absorbing shock, are needed in place of expensive grillage which is going to suffer more in the future than in the past. Roads cannot be designed to prevent accidents caused by drivers following too closely, which are likely to involve a series of cars. Also, is it not time to forget the emphasis on minor over-speeds of the decent driver and concentrate on the driver who by his behavior creates accident hazards? The driver who passes in a no-passing zone, who parks his vehicle on the pavement, and who crowds the other fellow's lane is a killer. Why not take him off the road?

The purely routine speed checking is as futile and wasteful of enforcement officers' time and ability as is their detail to check on overtime parking, both of which only result in congestion in the traffic courts on matters having a minimum relation to real traffic safety. We need the time of the traffic officers and of the courts to take the dangerous driver off the roads. Only a very small percentage of drivers habitually disregard the rules of safe driving.

This matter of driver behavior goes very deep. It involves a psychology that may well be inherent in a people who have been, and are capable of, building and running the outstanding democracy of the world. It is the essence of the spirit of freedom which revolts against unfair or unreasonable restrictions but which is tempered in the large majority to support of the public welfare. So it seems reasonable to accept the pattern established by the behavior of the decent majority and deny the freedom of the road to the violator of this pattern. Safety of the highways is, as democracy is, a matter of cooperation and coordination.

(*Continued from page 153. Tables 14 and 15 are on page 155.*)

For each 1,000 trucks and truck combinations, the number of axles weighing more than the recommended limit was 42 in the Pacific region, 146 in the Middle Atlantic region, and 108 in the New England region. For truck combinations, corresponding figures are 114 for the Pacific region, 500 for the Middle Atlantic region, and 449 for the New England region. Eliminating the empty vehicles, there were in the Middle Atlantic and New England regions about two axle loads in excess of the recommended limit for each three loaded truck combinations.

Table 14 shows the number of vehicles per 1,000 vehicles of various types with an axle-group load in excess of the limits recommended by the A. A. S. H. O., and in excess of these limits by various percentages. For the country as a whole, of each 1,000 loaded and empty trucks and truck combinations, 25 had axle-group loads weighing in excess of the recommended limits, 12 of which exceeded the limits by over 10 percent. It was the truck combinations rather than the single-unit trucks that produced the high load concentrations since of each 1,000 combinations, 89 had axle-group loads weighing more than the recommended limits, of which 46 exceeded the limits by more than 10 percent.

Many vehicles were so loaded, of course, that they exceeded more than one recommended weight limit; and some vehicles had several axle loads weighing more than the recommended limits. Counting each vehicle only once, regardless of the number of particulars in which it exceeded the recommended limits, table 15 shows the number of vehicles per 1,000 (both loaded and empty included) of each type that exceeded the limits by various percentages. Those vehicles which exceeded more than one provision of the recommended restrictions were tabulated in the column showing the highest percentage excess for any item. In the United States as a whole, 59 vehicles out of every 1,000 were overloaded to some degree and 15 out of every 1,000 exceeded some one of the provisions by more than 20 percent.

The Effect of Tax Increases on Gasoline Consumption

BY THE DIVISION OF RESEARCH REPORTS
PUBLIC ROADS ADMINISTRATION

Reported by E. M. COPE, Chief, Highway Statistics Section, and L. L. LISTON, Assistant Highway Economist

The possible effect of increases in State gasoline-tax rates on the use of highways has always been a matter of concern to highway administrators and highway users. This article describes a study of the problem, considering tax-rate changes and their effects on gasoline consumption and motor-vehicle travel during the period January 1946–July 1948.

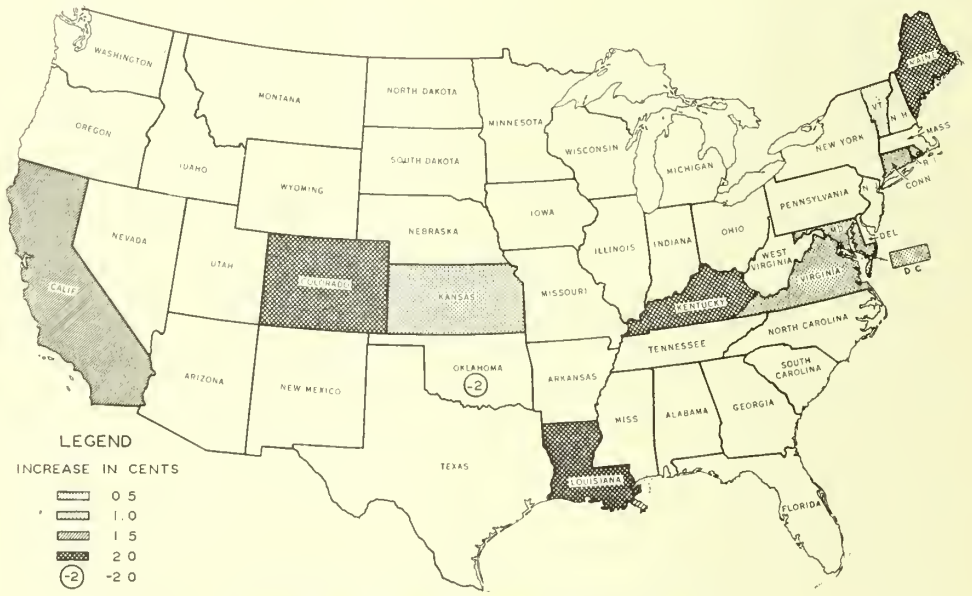
From an analytical standpoint, one of the principal difficulties encountered was the fact that there were parallel and much greater increases in the sale price of gasoline, excluding tax. The largest tax increase in any State in the 30-month period under study was 2 cents, while the smallest price increase was 3.5 cents. There were tax increases in only 12 States, but there were price increases in all States. Considerable fluctuation in gasoline sales was caused by "stocking up" just prior to the effective date of tax increases, with a consequent reduction in inventories immediately thereafter, thus creating the impression that the tax increase had reduced the consumption of gasoline.

It was found that consumption of gasoline in States where there were tax increases was similar in pattern to that in adjacent States where the tax was not increased. The increases in State gasoline tax rates, while causing substantial fluctuations in the sales of gasoline at the point at which it is taxed (the distributors, in most States), did not measurably affect net gasoline consumption. Sales price increases, since they received no advance publicity, had no apparent effect at all.

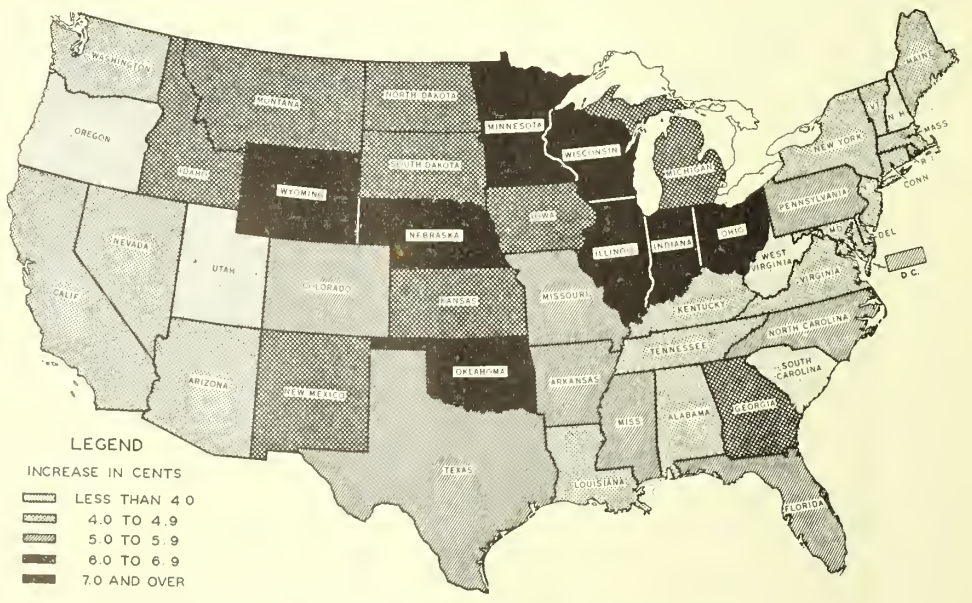
It is reasonably clear that at present price, wage, and employment levels, the demand for gasoline is so inflexible that no price or tax increases within reasonable contemplation will increase the total price to the consumer to the point where it will measurably affect the demand for highway gasoline.

From July 1, 1948, the close of the period covered by this study, to February 1, 1949, there were further net increases in the retail price of gasoline in 35 States, ranging from 0.1 to 1.5 cents; and two decreases of 0.1 and 0.2 cent. There were no increases in gasoline taxes.

THE FLEXIBILITY of the demand for gasoline is a problem that has long concerned highway economists and highway administrators. Stated in its simplest terms, the question is "do changes in the cost of gasoline to the consumer affect the quantity



INCREASES IN STATE GASOLINE TAX RATES



INCREASES IN GASOLINE PRICES EXCLUDING TAXES

Figure 1.—Increases in State gasoline tax rates and in gasoline sale prices excluding taxes, January 1, 1946–July 1, 1948.

of gasoline used on the highway?" The question is particularly important at this time. Many States have found that their current revenues from gasoline taxes—the revenues that, in large part, determine their ability to build and maintain an adequate highway system—are insufficient to construct and maintain highways to modern service and safety standards.

Since January 1, 1946, there have been 12 increases in State gasoline tax rates. There has also been one decrease—a temporary 2-cent tax in Oklahoma expired on January 1, 1947. The increases in State gasoline taxes are shown in table 1.

The purpose of this study was to determine, if possible, whether these changes in State gasoline-tax rates have affected the consumption of gasoline on the highways. The study is based entirely on the 30-month period from January 1946, through June 1948, and the discussion covers that period unless otherwise specified. The gasoline consumption and traffic comparisons made are all for the 1947–1946 and 1948–1947 relationships. Kansas and Virginia increased their tax rates from 3 to 4 cents and from 5 to 6 cents, respectively, in 1946, but since comparisons of relative changes in these two States would have necessitated use of 1945 data for periods in which gasoline rationing was still in effect, these two States were omitted from the study insofar as the effects of their 1946 gasoline-tax rate increases are concerned. The Louisiana tax rate increased from 7 to 9 cents per gallon on June 7, 1948, but sufficient data are not yet available to permit a full evaluation of the effects of that change. It is felt, however, that this study, based on the 1947 and 1948 changes in State tax rates, has a sufficiently broad base to justify its conclusions.

The principal difficulty encountered in this study was the fact that gasoline prices, excluding tax, increased rather sharply during the period under study, and in all cases the price increases were more than twice the amount of the increases in the tax rate.

In this article the word "price", unless otherwise noted, means the observed retail price of "regular" grade gasoline, excluding the State tax. These prices were obtained for the District of Columbia and for the capital city of each State, except in Maryland and Oregon, for which Baltimore and Portland prices were used. Since gasoline prices fall within a relatively narrow range in any area, it is felt that these prices are fairly representative. The gasoline consumption data used are from Public Roads analyses of motor-fuel consumption, which in turn are based on reports received from the State authorities. The traffic data were obtained from automatic traffic recorders located principally on rural roads in each State. Since the number of these recorders is limited, and comprehensive urban traffic information is not available, the data obtained from them for any one State must be considered as indicative rather than as an absolute measurement. The State-by-State traffic figures were, however, sufficiently uniform and consistent with the gasoline consumption data to warrant their use in the study. There is a probability that rural traffic in the period increased slightly more than urban traffic, but it does not appear that this could have been sufficient to make a significant difference in the findings.

Table 1.—*Increases in State gasoline taxes, January 1, 1946–July 1, 1948*

State	Tax rate			Effective date of change
	Old rate	New rate	Increase	
	Cents	*Cents*	*Cents*	
Kansas	3	4	1	Mar. 1, 1946
Virginia	5	6	1	June 19, 1946
Colorado	4	6	2	Apr. 1, 1947
Vermont	4	4.5	.5	May 1, 1947
Rhode Island	3	4	1	May 28, 1947
Maine	4	6	2	June 1, 1947
California	3	4.5	1.5	July 1, 1947
Connecticut	3	4	1	July 1, 1947
Maryland	4	5	1	July 1, 1947
Dist. of Columbia	3	4	1	Aug. 1, 1947
Kentucky	5	7	2	Apr. 1, 1948
Louisiana	7	9	2	June 7, 1948

PRICE RISES EXCEEDED TAX INCREASES

The January 1, 1946, price of regular gasoline and the State tax rates in effect on that date are given in table 2. This table shows the month-by-month changes in prices and taxes that took place between January 1, 1946, and July 1, 1948, and gives a summary of the cumulative amounts of those changes. The price increases in each case represent the increase in the reported price on the first day of the month over the reported price on the first day of the month preceding; thus, the increases shown for August are the amounts by which August 1 prices exceeded July 1 prices. The changes in tax rates are listed under the month in which they occurred, since the exact dates are known.

There were two gasoline-tax rate changes in 1946—Kansas and Virginia each increased their tax rates 1 cent. Between July 1 and September 1 of the same year the gasoline prices in 36 States were increased the same amount or more, and between December 1, 1947, and January 1, 1948, there was a substantial price increase in every State. The smallest of these increases was 0.9 cent and the largest was 2.5 cents. The latter increase occurred in Iowa, and it is worth noting that it was attended by nowhere near the publicity that had been accorded the increase of 1 cent in the State's tax rate that occurred in 1945. In addition to the Iowa price increase of 2.5 cents, there were price increases of 2 cents per gallon or more in 17 other States between November 1, 1947, and January 1, 1948. During the period in which most of the tax increases occurred—April 1 through July 1, 1947—there were relatively few increases in the price of gasoline. This, of course, had a tendency to silhouette the tax changes and they received a considerable amount of unfavorable publicity.

Figure 1 shows the increases in gasoline taxes and the price increases in the period under study. As may be seen in the upper map, there was no geographic concentration in the tax increases. It is worth mentioning, however, that beginning with Massachusetts, New York, and New Jersey, and continuing westward through the populous and highly developed North Central and Central States, the 3-cent and 4-cent taxes in that area remained lower than the rates generally in effect throughout other parts of the country. It is in this group of States that the greatest agitation for toll roads is found. The tremendous latent opposition to increases in the gasoline-tax rates in these States has probably contributed a great deal towards forcing the proponents of high-standard highways to turn to toll roads as the only alternative offering some chance of obtaining modern highways within a relatively short period of time. This rigidity in the gasoline-tax rates in the area is encouraging support for toll roads that would have approximately the same economic effect on the user as 15 to 19 cents per gallon gasoline-tax rates on certain types of traffic.

Although it can be observed from the upper map in figure 1 that there was no geographic significance to the increases in the State gasoline-tax rates, the lower map in the figure shows a remarkable concentration of the larger price increases beginning with the North Central States and continuing as far west as Idaho and southwest to New Mexico. In fact, the very large increases in the price of gasoline in many States exceeded, in total cost to the consumer, the combined price and tax increases in other States. In Oklahoma, where there was a decrease of 2 cents in the tax rate during the period, the price of gasoline increased 8 cents per gallon, resulting in a net increase in cost to the consumer of 6 cents per gallon.

PRICE RISES GREATER WHERE TAX WAS UNCHANGED

The difficulty of determining the effect of gasoline-tax increases was accentuated by the fact that the average price increases in the States in which there were also tax increases was 4.53 cents per gallon, whereas in the States where the tax remained the same throughout the period, the price of gasoline was increased an average of 5.61 cents per gallon. The more than 1-cent-per-gallon greater average increase in price in the States in which there were no tax increases imposed an additional burden on consumers that, in theory at least, would have the same restraining effect as the tax increases, thus making it difficult to determine the economic effects of tax increases. The greater price increases in the States that did not have tax increases might possibly have been explained by economic factors in fields not explored during the study. However, available data and facilities made impractical an effort to isolate such factors. Furthermore, it did not seem that additional investigation on this point would make a material contribution toward the objective of determining whether State gasoline-tax rates had affected highway gasoline consumption.

Figure 2 shows for each State the total price and tax changes in the 30-month period. There was one decrease in tax rate and there

Table 2.—*Gasoline price and tax changes by months, Jan. 1, 1946-July 1, 1948*

[*In cents*]

State	Price of gasoline excluding State tax Jan. 1, 1946 [1]	State gasoline tax Jan. 1, 1946	Changes during 1946																				Changes during 1947							
			February		March		April		May		June		July		August		September		October		November		December		January		February		March		April	
			Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax
Alabama	18.5	6.0																							0.5						1.5	
Arizona	17.5	5.0													1.0										0.5		0.5				0.8	
Arkansas	15.5	6.5													1.0																2.0	
California	16.5	3.0													1.0												0.5				0.8	
Colorado	16.0	4.0													0.5		0.5														1.1	2.0
Connecticut	16.0	3.0			−0.5				0.2		0.2		0.1		0.9										0.2						1.1	
Delaware	15.3	4.0											0.3		1.0						0.3		0.1								1.4	
Florida	16.0	7.0			−0.2								0.2				0.8								0.3						1.2	
Georgia	15.5	6.0													0.5		0.8												0.3		0.9	
Idaho	19.0	6.0													1.0												0.5				0.8	
Illinois	15.7	3.0													1.1		0.9										0.3				1.0	
Indiana	15.9	4.0	−0.3								0.3		1.1				0.5								0.3				0.7		0.5	
Iowa	14.9	4.0					−0.2						0.2				1.7				0.5								0.2		1.0	
Kansas	14.9	3.0				1.0											2.1														1.1	
Kentucky	17.5	5.0	−0.2								−1.3				1.0								0.3		0.1				0.3		0.8	
Louisiana	15.5	7.0													0.5								0.5				0.5		1.0			
Maine	15.5	4.0			−0.9				0.9						0.9		0.4								0.2						1.1	
Maryland	14.2	4.0			−0.3				0.3								1.0						0.3						0.3		1.0	
Massachusetts	14.7	3.0			−0.1				0.1						0.9		0.5								0.2						1.1	
Michigan	15.9	3.0	−0.3										0.3		1.1		0.5														1.0	
Minnesota	15.2	4.0	−0.1		−0.1		0.1		−0.2		0.3		0.2		−0.8		2.0		−0.1		0.1		0.3						0.3		0.5	
Mississippi	14.8	6.0	−0.2								0.3												1.1						0.4		0.8	
Missouri	15.7	2.0									0.5						0.5										0.4		0.5		1.0	
Montana	18.0	5.0													2.0				1.0										1.0		0.2	
Nebraska	13.5	5.0													1.4		0.1		1.0				0.6		−0.1		−1.0		0.5		1.0	
Nevada	19.0	4.0													1.0												1.0		0.8			
New Hampshire	17.2	4.0															1.0						0.3								1.0	
New Jersey	15.5	3.0													1.0								0.3								1.1	
New Mexico	17.0	5.0													1.0		0.5				0.5										1.5	
New York	15.0	4.0					0.2				0.1				1.0		0.2								0.2		0.4				0.7	
North Carolina	16.2	6.0					0.6		−1.2						0.5								0.9						1.0		1.0	
North Dakota	17.7	4.0	−0.3												1.6		0.5										0.1				1.0	
Ohio	14.0	4.0																									1.5				1.5	
Oklahoma	13.5	7.5					1.0				−1.0		−1.0		1.5		2.5									−2.0						
Oregon	16.5	5.0													1.0										−0.5		0.5				0.8	
Pennsylvania	14.7	4.0													0.3		1.5								0.8						1.1	
Rhode Island	14.8	3.0													0.9		−0.1						0.2		0.2						1.1	
South Carolina	17.8	6.0							0.3														0.5								0.8	
South Dakota	17.5	4.0															1.8										0.2		0.2		0.3	
Tennessee	16.0	7.0													0.5		0.5								0.5						1.1	
Texas	14.5	4.0													1.0														0.5		0.5	
Utah	19.5	4.0															1.0		−0.5												1.0	
Vermont	17.4	4.0							−0.1		0.2		0.1		1.0				−0.1				0.1		0.6						1.1	
Virginia	15.6	5.0												1.0			0.8												0.7		0.9	
Washington	16.5	5.0															1.0										0.5				0.8	
West Virginia	18.0	5.0	−0.4		−0.3										0.6								0.3						0.2		0.8	
Wisconsin	15.9	4.0							−0.3				0.3		1.3		0.8										0.5				1.0	
Wyoming	16.0	4.0													1.0								2.0									
Dist. of Columbia	14.5	3.0			−0.3										1.3										0.3				0.3		1.0	

[1] The observed retail price of regular gasoline, including Federal and local taxes but excluding State taxes.

Table 2.—*Gasoline price and tax changes by months, Jan. 1, 1946–July 1, 1948—Continued*

[in cents]

State	Changes during 1947, continued																Changes during 1948														Total for period Jan. 1, 1946–July 1, 1948	
	May		June		July		August		September		October		November		December		January		February		March		April		May		June		July			
	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax	Price	Tax
Alabama															1.0		1.0				0.5										4.5	
Arizona			−0.3												0.2		1.8														4.6	
Arkansas											0.5						1.5												0.1		5.2	
California			−0.3		1.5										0.1		1.8												0.3		4.2	1.5
Colorado	0.9												1.0		−0.6		1.9														5.3	2.0
Connecticut					1.0								0.3				1.5														4.0	1.0
Delaware											0.3				0.3		1.0														4.7	
Florida	0.2		0.5				0.5						0.5				1.0				0.5										5.5	
Georgia			0.5						0.5		−0.4		0.4		1.0		1.0				1.0										6.5	
Idaho	0.7		−0.3												0.5		1.8												1.1		6.1	
Illinois													1.1		0.5		2.0		0.3								0.2				7.4	
Indiana							0.4						1.3				1.5		0.6								0.1				7.0	
Iowa													0.2				2.5		0.5								0.1				6.7	
Kansas					0.4								0.3		0.1		2.1														6.1	1.0
Kentucky			0.5												1.0		1.0				0.5		2.0								4.0	2.0
Louisiana													0.5				1.5												2.0		4.5	2.0
Maine				2.0			0.2								0.3		0.9														4.0	2.0
Maryland					0.2	1.0							0.3		0.4		0.9		0.1				0.1		0.1		−0.1		0.1		4.7	1.0
Massachusetts							0.5								0.3		1.4														4.9	
Michigan	0.5								0.7		0.1		1.1		0.1		1.5				0.1						0.1				6.8	
Minnesota	0.2		−0.2		−0.8		2.3		−0.8		0.3		1.2		1.1		2.0		0.3												8.1	
Mississippi	0.5		0.1				0.1				1.2		−2.0		0.6		2.0				0.4						0.1				5.4	
Missouri													1.1				1.5		0.2								0.2				5.9	
Montana													0.9				1.4		0.1												6.6	
Nebraska	0.5				0.7				0.1				0.7				2.0		−0.3												7.2	
Nevada			−0.3												0.3		1.8												0.3		4.9	
New Hampshire							0.1								0.3		0.9														3.6	
New Jersey											−0.3		0.3		0.6		1.0														4.0	
New Mexico													1.5				1.7												−0.2		6.5	
New York							0.3										1.4														4.5	
North Carolina																	2.2														5.0	
North Dakota									0.5				1.2		0.1		1.6		0.1								0.3				6.7	
Ohio					1.0								1.0				1.5												1.0		7.5	
Oklahoma	1.0												1.0				2.0										0.5				7.5	−2.0
Oregon			−0.3												0.2		1.8														3.5	
Pennsylvania															0.6		1.0														5.3	
Rhode Island			0.2	1.0			0.7								0.4		1.0										−0.2		0.2		4.6	1.0
South Carolina	0.1								0.5		−0.5		0.5				1.2														3.4	
South Dakota	0.5												1.8				0.9		0.2								0.1				6.0	
Tennessee									0.9						−0.3		1.3														4.5	
Texas			0.5				0.5								0.5		1.5														5.0	
Utah	1.0																1.0														3.5	
Vermont		0.5			0.8								0.4				0.9														4.8	0.5
Virginia													0.5				1.2														4.1	1.0
Washington			−0.3												1.1		1.8														4.9	
West Virginia									0.5				0.3				1.4														3.4	
Wisconsin													1.5				2.0		0.5								0.1				7.7	
Wyoming	1.5				1.0								1.0				1.5														8.0	
Dist. of Columbia					1.0			1.0					0.3				1.0		0.3												5.2	1.0

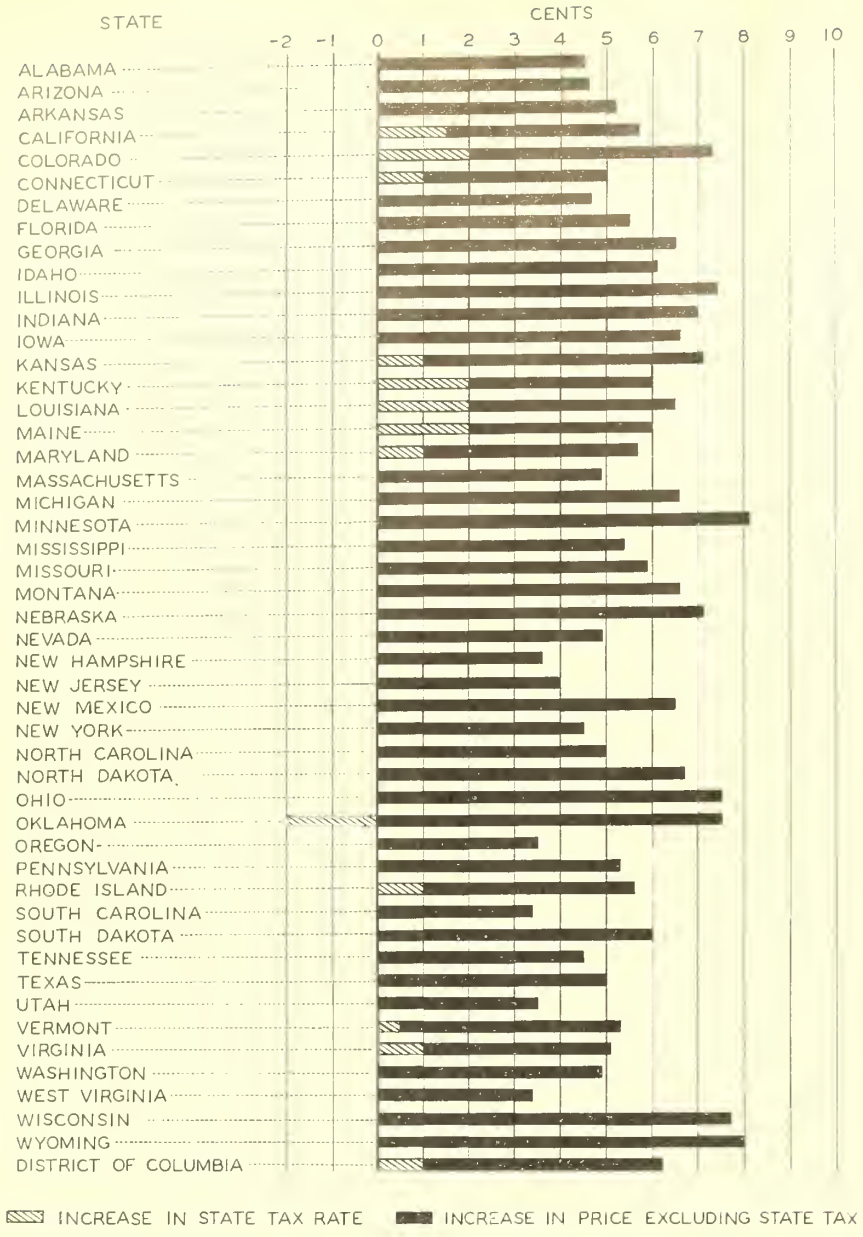


Figure 2.—Gasoline price and tax increases, January 1, 1946–July 1, 1948.

were no decreases in price. The increases in tax rates ranged from 0.5 cent per gallon in Vermont to 2 cents per gallon in Colorado, Kentucky, Louisiana, and Maine; and the increases in price ranged from 3.4 cents per gallon in South Carolina and West Virginia to 8.1 cents per gallon in Minnesota.

Table 3 shows the relative changes in sales of gasoline for use on highways, and the changes in traffic, for the 2 months prior to the month in which the change in tax rate took place, the month in which the tax rate took effect, and the following 2 months. The figures given show the changes, in percent, of 1947 sales and traffic compared to the same months of 1946 in all cases except for Kentucky and Louisiana. The latter two States increased their taxes in 1948, and the comparisons are therefore of 1948 data to those for 1947. Louisiana is omitted from table 3 because the tax change there was so recent that sufficient data were not available at the time the study was made. The Rhode Island tax increase from 3 to 4 cents took effect May 28, and for practical purposes was treated in this study as if it had taken place in June.

Within each group in table 3, the State in which the tax increase occurred is listed first, followed by adjoining States in which there were no changes in tax rates. One exception to this is the inclusion of Vermont for comparison with Maine. There was a half-cent-per-gallon increase in the Vermont tax on May 1, 1947, but it was felt that this increase did not destroy the validity of the comparison since the Maine tax increase a month later was 2 cents per gallon. It was not expected that the half-cent increase in the Vermont tax would be reflected in gasoline sales, and the State therefore was not set out in a separate grouping; but the table does show a very noticeable fluctuation in Vermont highway gasoline sales.

PUBLICITY ON TAX INCREASES INDUCES "LOADING UP"

Although any economic effects of gasoline-tax increases, as well as price increases, must be continuing, it seemed a reasonable assumption at the time the study was undertaken that the impact of a tax increase, reflected in the increased cost to the consumer, would be distinguishable in the demand for gasoline at the time the tax was imposed. This seemed especially likely because of the very great publicity given to the tax increases. The significance of table 3 therefore lies more in the month-by-month figures than in the totals for the 5-month periods bracketing the tax increases. A very noticeable "loading up" can be observed in the months prior to the tax increases, coupled with a reduction in inventories in the month of change and the succeeding month. Rhode Island appears to be an exception, but this probably was the result of a combination of factors: Gasoline sales in the same month of the prior year were abnormally low; Rhode Island is a small State whose highways carry a relatively high proportion of interstate traffic; and the tax rate before the increase was only 3 cents. Furthermore, it should be noted that Rhode Island traffic in the month in which the change took place was up 17.4 percent over the same month of the prior year, and in the month following was up more than 40 percent; but it is also possible that the traffic-recorder data in this instance did not accurately reflect actual changes in traffic.

The gasoline tax is imposed on the "first sale" in most States. Consequently, the taxed sales given in table 3 reflect any changes in distributors', retailers', and consumers' inventories and do not necessarily reflect the month of the use of gasoline on the highways by the ultimate consumer. (The gasoline consumption data in table 4, however, have been adjusted to the month of consumption, and in this respect differ from those in table 3.)

In most States the tax on gasoline applies to distributors' in-shipments, and in the remaining States to distributors' sales. The dates of tax increases are known in advance and there appears to be a strong tendency to utilize all storage capacity immediately prior to the tax increases. Although tax increases are frequently accompanied by "floor taxes," imposing the additional tax on inventories, these increased inventories reveal, to a degree, the hope of somehow avoiding the tax. Undoubtedly this "loading up" takes place throughout the supply channels, including retail outlets, and probably includes a considerable amount of gasoline sold to consumers who purchase in bulk. In Maine, for instance, the May 1947, sales of gasoline for highway use exceeded those of May 1946, by 28.3 percent, but gasoline sold in June, the month of the tax change, was 11.5 percent less than in the same month of 1946. Traffic, on the other hand, was 8 percent greater in May 1947, and 9.8 percent greater in June. The 9.8 percent increase in June 1947, over June 1946, was, as a matter of fact, greater than in the adjoining States of New Hamp-

Table 3.—*Percentage changes in highway gasoline sales and traffic in year of tax increase from same months of prior year*

Month in which tax was increased	State	Second preceding month		Preceding month		Month of change		Following month		Second following month		Total, 5 months	
		Highway gasoline sales	Traffic	Highway gasoline sales	Traffic	Highway gasoline sales	Traffic	Highway gasoline sales	Traffic	Highway gasoline sales	Traffic	Highway gasoline sales	Traffic
Apr. 1947	Colorado	+14.7	+0.5	+19.1	+6.4	−2.9	+5.9	+3.3	+8.5	+7.9	+12.3	+8.1	+7.0
	Oklahoma	+12.9	+2.0	+11.5	+2.2	+2.1	+6.8	+9.1	+2.9	+11.5	+7.5	+9.3	+4.4
	Utah	+15.9	+5.6	+15.1	+11.1	+4.2	+6.7	−0.8	+13.5	+9.9	+10.8	+8.2	+9.8
	Wyoming	+19.6	+4.0	+21.9	+13.6	+7.5	+5.5	+8.6	+9.2	+19.2	+12.5	+14.8	+9.2
	Group total	+14.3	+2.0	+15.2	+5.2	+1.4	+6.4	+5.7	+6.1	+10.9	+9.7	+9.2	+6.1
	U. S. total	+11.6	+8.8	+7.1	+6.0	+3.5	+9.7	+12.5	+10.6	+13.8	+11.3	+9.7	+9.4
June 1947	Maine	+10.3	+5.3	+28.3	+8.0	−11.5	+9.8	+4.5	+9.7	+7.9	+13.0	+7.6	+9.4
	New Hampshire	+1.0	+6.3	+8.0	+10.8	+10.4	+5.0	+8.8	+12.0	+11.4	+15.1	+8.4	+10.4
	Vermont	+14.4	+7.6	−3.1	+10.4	+12.2	+7.1	+6.7	+6.2	+8.6	+12.9	+7.5	+8.9
	Group total	+8.5	+6.1	+16.0	+9.2	−0.5	+8.0	+6.2	+9.4	+9.1	+13.5	+7.8	+9.5
	Rhode Island	+2.9	+7.2	+4.4	+16.4	+12.0	+17.4	+12.5	+40.7	+8.7	+26.1	+8.2	+22.6
	Massachusetts	+5.5	+8.9	+6.9	+11.6	+9.8	+6.8	+4.1	+7.5	+12.6	+13.8	+7.8	+9.8
	New Jersey	+5.5	+14.8	+7.8	+10.4	+12.3	+8.0	+2.1	+11.0	+8.0	+13.2	+7.1	+11.1
	New York	+1.9	+8.5	+7.7	+8.2	+8.9	+14.1	+10.8	+22.0	+8.2	+9.0	+7.7	+12.7
	Group total	+3.5	+9.8	+7.4	+9.5	+10.0	+11.7	+7.4	+18.2	+9.0	+11.1	+7.6	+12.3
	U. S. total	+3.5	+9.7	+12.5	+10.6	+13.8	+11.3	+7.4	+11.7	+8.0	+10.9	+9.0	+10.9
July 1947	California	+3.1	+8.0	+18.1	+6.1	+1.2	+4.5	+11.4	+5.3	+17.3	−2.2	+10.1	+4.1
	Nevada	+5.6	+6.1	+13.8	+10.2	+14.1	+11.3	+6.6	+12.3	+7.8	+12.5	+9.6	+10.6
	Oregon	+14.7	+9.1	+11.1	+14.1	+15.8	+17.5	+7.5	+19.6	+10.4	+12.9	+11.8	+14.8
	Washington	+6.3	+4.6	+10.5	+7.1	+9.1	+2.2	+4.7	+6.5	+7.4	+4.7	+7.5	+5.0
	Group total	+4.9	+7.5	+16.1	+7.4	+4.2	+6.0	+9.8	+7.5	+14.9	+1.2	+9.9	+5.9
	Connecticut	+10.5	+11.1	+21.8	+13.1	−0.3	+15.6	+6.7	+16.3	+12.1	+12.1	+9.9	+13.8
	Massachusetts	+6.9	+11.6	+9.8	+6.8	+4.1	+7.5	+12.6	+13.8	+8.0	+8.1	+8.2	+9.5
	New Jersey	+7.8	+10.4	+12.3	+8.0	+2.1	+11.0	+8.0	+13.2	+21.1	+9.1	+9.8	+10.3
	New York	+7.7	+8.2	+8.9	+14.1	+10.8	+22.0	+8.2	+9.0	+16.4	+5.8	+10.3	+11.8
	Group total	+7.8	+9.4	+11.1	+11.7	+6.5	+17.3	+8.8	+11.1	+15.3	+7.2	+9.8	+11.3
	Maryland	+16.2	+10.8	+19.6	+11.3	+1.2	+8.4	+3.7	+8.4	+7.8	+3.0	+9.4	+8.3
	Delaware	+7.3	+19.3	+8.1	+22.2	+6.7	+18.8	+2.4	+17.3	+9.1	+5.4	+6.6	+16.9
	Pennsylvania	+8.3	+8.7	+12.4	+10.8	+4.8	+12.1	+7.2	+12.1	+13.4	+10.4	+9.0	+10.9
	Virginia	+13.4	+10.7	+10.7	+13.9	+11.2	+13.9	+0.5	+14.5	+12.4	+7.7	+9.4	+12.1
	Group total	+10.3	+10.3	+12.9	+12.5	+5.7	+12.3	+5.2	+12.4	+12.3	+8.1	+9.1	+11.1
	U. S. total	+12.5	+10.6	+13.8	+11.3	+7.4	+11.7	+8.0	+10.9	+13.7	+8.3	+11.0	+10.6
Aug. 1947	Dist. of Columbia	+10.4	+5.3	+16.3	+9.3	+5.4	+4.2	+11.7	+3.9	+6.6	+18.4	+10.0	+6.0
	Delaware	+8.1	+22.2	+6.7	+18.8	+2.4	+17.3	+9.1	+5.4	+9.5	+11.3	+7.0	+15.5
	Pennsylvania	+12.4	+10.8	+4.8	+12.1	+7.2	+12.1	+13.4	+10.4	+13.0	+15.0	+10.0	+12.2
	Virginia	+10.7	+13.9	+11.2	+13.9	+0.5	+14.5	+12.4	+7.7	+28.0	+10.2	+11.9	+12.1
	Group total	+11.7	+11.2	+6.9	+12.7	+5.4	+12.4	+13.0	+8.4	+15.5	+13.1	+10.3	+11.6
	U. S. total	+13.8	+11.3	+7.4	+11.7	+8.0	+10.9	+13.7	+8.3	+11.1	+8.9	+10.7	+10.2
Apr. 1948	Kentucky	+0.1	+21.1	+44.0	+38.5	−5.5	+21.1	+11.6	+21.4	+12.3	+20.7	+12.4	+24.3
	Ohio	+20.7	+12.6	+12.3	+14.9	+11.7	−0.4	+17.1	+5.2	+5.0	+10.7	+12.8	+8.2
	Tennessee	+1.1	+7.5	+7.6	+16.7	+11.1	+14.9	+24.6	+11.8	−4.3	+13.6	+8.1	+12.9
	West Virginia	+13.4	+14.0	+27.4	+23.6	+24.2	+7.4	+24.9	−1.5	+9.0	+10.7	+19.4	+9.8
	Group total	+13.3	+13.4	+17.6	+20.7	+10.2	+7.0	+18.3	+8.3	+4.9	+12.9	+12.5	+12.2
	U. S. total	+6.2	+5.5	+10.9	+7.9	+10.6	+7.9	+5.0	+8.4	(1)	+7.1	(1)	+7.4

[1] Data not available at time this table was prepared.

shire and Vermont, where increases of 5.0 percent and 7.1 percent, respectively, were experienced. In Connecticut, gasoline sold for highway use in June 1947, exceeded that for June 1946, by 21.8 percent, but in July it was 0.3 percent less than that for July of the previous year. For the 5-month period Connecticut highway gasoline sold showed a 9.9 percent increase in 1947 over the same months of 1946, slightly greater than the average for the adjoining States—Massachusetts, New York, and New Jersey.

The immediate effect of this "loading up" is to give the appearance, in the month of the tax increase and immediately following while inventories are being reduced to normal, that gasoline consumption has suffered substantially as a result of the impact of the tax increase. A close analysis of gasoline consumption, however, reveals that the 5-month totals for the States in which the increases occurred are very similar to those of the adjoining States in which there were no increases. As inventories in the supply channels are brought back to normal it frequently appears, to those not thoroughly familiar with the distribution and taxation of gasoline, that the demand for gasoline has been decreased by the increase in tax. This misinterpretation has frequently accounted for articles appearing in newspapers and trade journals to the effect that gasoline demand has been sharply cut by tax increases. Recent instances of such misinterpretation occurred with respect to Vermont and Kentucky gasoline consumption. In both cases abnormally large amounts of gasoline had been purchased immediately prior to the imposition of the tax increase, but the total amount sold in the 5-month period bracketing the increases showed no marked deviation between the sales patterns of those States and those of adjoining States.

TRAFFIC UNAFFECTED BY TAX INCREASES

Table 3 also demonstrates that in each of the States in which the tax increases occurred the traffic maintained fairly normal patterns similar to those of adjoining States, and, allowing for regional differences, consistent with the pattern of total traffic in the United States. It will be noted that in each of the States in which tax changes occurred, traffic remained on a fairly even keel, in each case being similar in pattern to the traffic of adjoining States. This appears to be conclusive evidence that the impact of the tax increases, while resulting in a violent fluctuation in gasoline sold, does not substantially affect the use of gasoline on the highways.

Theoretically, each increase in the cost of gasoline, whether arising from taxes or other factors, should reduce the potential market; and each decrease in the cost should expand the potential market. In a period of relatively stable prices and employment it might have been possible to distinguish the effects of the changes in gasoline-tax rates, or conversely to decide with a reasonable degree of certainty that the changes had no appreciable effect on consumption. The fact that there were 12 increases in tax rates and 1 decrease during the period under study would in normal times have created, as nearly as it is possible to create, a laboratory condition. It might have been possible to have settled once and for all the question of the extent to which the demand for gasoline for highway use is flexible, and perhaps to have made a determination of its sensitivity. From the point of view of making this determination, it is unfortunate that the tax increases were, to a considerable degree, the result of a general

rise in all price levels that at the same time rendered comparisons difficult.

There can be no doubt that at some point the cost of highway transportation will govern the amount of gasoline used, and, for that matter, the distribution and ownership of motor vehicles; but it appears that we have not yet arrived at the point where these costs are sufficient to constitute an effectual brake on the use of highway transportation. Consumption of gasoline under present conditions of full employment and high wages is more likely to be determined by the number of vehicles available and the facilities for their use than by the price of gasoline. Gasoline and oil costs, which account for the principal day-to-day expense of operating an automobile, amount to approximately 1.8 cents per mile. If we take, for example, a new automobile costing the consumer $1,800 and assume that it is to be depreciated over 100,000 miles, the depreciation alone is 1.8 cents per mile, equaling the cost of gasoline and oil, and allowing nothing for maintenance, garaging, insurance, registration fees, and property taxes.

It is quite possible that the economic effects of high motor-vehicle transportation costs will be revealed much sooner in the market for automobiles than in the market for gasoline. Even allowing for the fact that the average consumer may buy, or refrain from buying, gasoline at any given time and that the demand for gasoline, therefore, must be much more sensitive than the demand for cars, and the fact that the automobile market must, of necessity, have much less short-term sensitivity, it seems probable that any change in the long-term curve of gasoline demand will be preceded by a very substantial weakening in the market for automobiles. In other words, it does not appear that the need for highway transportation has been satisfied to the point that gasoline price increases or tax increases, within reasonable limits, will have any effect on the amount of gasoline consumed.

GASOLINE COST INCREASE LESS THAN INCREASE IN GENERAL LIVING COST

On January 1, 1946, the unweighted arithmetic average retail price of gasoline for the United States was 20.6 cents per gallon. At the end of the 30-month period under study the same average price had risen to 26.3 cents, an increase of 27 percent. In the same period the Bureau of Labor Statistics' Cost of Living Index, based on the 1935–39 period, rose from 129.9 in January 1946, to 173.7 in July 1948, an increase of more than 33 percent. Thus, despite the fact that the total cost of gasoline to the consumer rose rapidly during the period, its relative increase was not as great as the increase in his general cost of living. Between January 1946, and July 1948, hourly earnings in manufacturing industries increased from an average of just over $1.00 an hour to more than $1.30 per hour. Agricultural incomes increased even more rapidly, and there were substantial increases in other industries, coupled with full employment. During this period the relative cost of gasoline, including taxes, actually decreased for all groups except those depending on fixed annual salaries, annuities, rents, and other relatively stable sources of income. It was with these facts in mind that this inquiry was limited to an effort to determine whether the increases in State gasoline taxes had affected gasoline consumption, rather than an attempt to learn under what conditions gasoline consumption would have been affected.

Table 4.—*Highway use of motor fuel, 1946–47, in States that did and did not have tax increases*

Month	States in which tax rate increased during 1947			States in which tax rate did not increase during 1947			Total, all States		
	1946	1947	Change	1946	1947	Change	1946	1947	Change
	1,000 gallons	*1,000 gallons*	*Per-cent*	*1,000 gallons*	*1,000 gallons*	*Per-cent*	*1,000 gallons*	*1,000 gallons*	*Per-cent*
January	271,130	327,986	+21.0	1,483,602	1,695,510	+14.3	1,754,732	2,023,496	+15.3
February	266,803	312,988	+17.3	1,421,094	1,571,311	+10.6	1,687,897	1,884,299	+11.6
March	311,059	320,428	+3.0	1,691,364	1,824,031	+7.8	2,002,423	2,144,459	+7.1
April	320,213	343,085	+7.1	1,888,298	1,943,300	+2.9	2,208,511	2,286,385	+3.5
May	344,709	366,271	+6.3	1,894,852	2,153,106	+13.6	2,239,561	2,519,377	+12.5
June	340,949	395,062	+15.9	1,859,000	2,108,651	+13.4	2,199,949	2,503,713	+13.8
First 6 months	1,854,863	2,065,820	+11.4	10,238,210	11,295,909	+10.3	12,093,073	13,361,729	+10.5
July	371,860	382,945	+3.0	2,061,328	2,230,541	+8.2	2,433,188	2,613,486	+7.4
August	371,261	404,741	+9.0	1,999,159	2,155,644	+7.8	2,370,420	2,560,385	+8.0
September	336,380	386,444	+14.9	1,818,374	2,062,616	+13.4	2,154,754	2,449,060	+13.7
October	346,408	388,972	+12.3	1,948,165	2,159,481	+10.8	2,294,573	2,548,453	+11.1
November	322,489	368,921	+14.4	1,839,408	1,882,730	+2.4	2,161,897	2,251,651	+4.2
December	326,750	396,911	+21.5	1,814,343	2,034,030	+12.1	2,141,093	2,430,941	+13.5
Last 6 months	2,075,148	2,328,934	+12.2	11,480,777	12,525,042	+9.1	13,555,925	14,853,976	+9.6
Total	3,930,011	4,394,754	+11.8	21,718,987	23,820,951	+9.7	25,648,998	28,215,705	+10.0

A summary of highway gasoline consumption is given in table 4. The 1946–47 changes are given in totals for the group of States in which there was a tax-rate increase in 1947, and in totals for the group of States in which the rate did not increase during 1947. These summaries show that in the first 6 months of 1947 the States in which the taxes were increased during the year had an increase of 11.4 percent in highway gasoline sales; and in the last 6 months of 1947 showed an even greater increase of 12.2 percent over the prior year. This is surprising, since most of the tax changes took place about the middle of the year, with most of the "loading up" of inventories taking place during the first 6 months, and the unloading during the last 6 months. Had it not been for the loading effect, the last-half increase in 1947 over the same period in 1946 would have been greater. Strangely enough, the States in which there were no increases in tax rates during 1947 showed an increase of 10.3 percent in highway gasoline consumption in the first 6 months of the year over the same months of 1946, and a smaller increase, 9.1 percent, in the last 6 months of the year. It is not possible to arrive at a conclusive explanation for this apparent defiance of economic law, if we consider the changes in tax rates alone. We do know, however, that the average price of gasoline, without tax, increased more in the States that did not increase the tax rate than it did in the States where the tax rates were increased, and for longer periods this undoubtedly nullified the relative effects of the tax increases.

CONCLUSIONS

It would be irresponsible to conclude from the data compiled that increased costs to the consumer would not, in the long run, have a tendency to reduce the consumption of gasoline. It is apparent, however, that we have not yet reached the point where this effect can be measured. At the time the study was undertaken it was hoped the effects of gasoline-tax increases could be measured, despite the fact that the price increases so overshadowed them. It was felt that the impact of the tax increases, considering the publicity given them, could be located and measured; but this expectation was not realized.

It is reasonably clear that at present price, wage, and employment levels, the demand for gasoline is so inflexible that no price or tax increases within reasonable contemplation will increase the total price to the consumer to the point where it will measurably affect the demand for highway gasoline.

New Publications

Work of the Public Roads Administration, 1948, the annual report of the Public Roads Administration for the fiscal year ended June 30, 1948, is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 20 cents a copy.

Highway Statistics, 1947, is also now available. The third in an annual series, it presents information of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, highway finance, and highway mileage for the year 1947. A summary bulletin reporting similar information over periods of 20 to 50 years, up to 1945, was printed last year. These publications are for sale by the Superintendent of Documents at the following prices:

Highway Statistics, 1947, 45 cents.

Highway Statistics, 1946, 50 cents.

Highway Statistics, 1945, 35 cents.

Highway Statistics, Summary to 1945, 40 cents.

Traffic Trends on Rural Roads in 1947

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by THOMAS B. DIMMICK, Highway Economist

Total rural-road traffic in 1947 broke all previous records, exceeding the previous high in 1946 by almost 10 percent. Peak summer travel was 7 percent greater than in 1941, previous record year for that season.

Travel on the main rural roads of the country amounted to more than 137 billion vehicle-miles, of which almost 80 percent was by passenger cars. Ton-mileage carried by commercial vehicles increased 21 percent for the second consecutive year. Reversing a previous trend, a slightly larger percentage of these vehicles were found to be carrying loads in 1947. Average carried loads remained about the same as in 1946.

The use of heavy commercial vehicles continued to increase, truck-combination travel being 21 percent greater than in 1946. The rise in frequency of heavy gross weights has apparently been halted, but the frequency of heavy axle loads was almost one-fifth greater in 1947 than in the previous year.

TRAFFIC on rural roads in 1947 broke all records, exceeding the all-time high of the previous year by almost 10 percent. Travel in each month of 1947 was greater than in the corresponding month of 1946, the increases ranging from 6 percent in March, November, and December to 13 percent in January. The peak summer travel was about 7 percent above that for 1941, the year of the previous all-time high for that season.

On the 345,000 miles of main rural roads of the country, travel in 1947 was more than 137 billion vehicle-miles, of which almost 21 percent was by trucks and truck combinations.[1] Vehicle-mileage of these commercial vehicles in 1947 increased to 18 percent above the 1946 figure and ton-mileage, 21 percent. The increased use of truck combinations, particularly noticeable during the war, was continued through 1947. The percentage of vehicles carrying loads increased slightly.

The total rural-road traffic data reported in this analysis are derived largely from records received from about 660 automatic traffic recorders operated continuously throughout the year at permanent stations on rural roads in all 48 States. These machine counts provide no classification by vehicle type since they record only the total number of all types passing. Trends in type, weight, and characteristics of trucks and truck combinations were obtained from a summer survey described later in this article. Supplemental counts made by several States yielded valuable information concerning the total volume of rural traffic within their boundaries and its classification by types of vehicles. Consideration has been given to all such available data, and those derived from the sample with the most complete coverage were given preference in this analysis. In instances where States have prepared and submitted vehicle-mile travel estimates, these have been employed rather than estimates made by applying trend factors.

Figure 1 illustrates the variation in rural traffic in the three main geographic divisions and in the United States as a whole in the year 1947; in the previous year, 1946; and in the peak prewar year, 1941. In the three eastern regions, 1947 traffic was less than that in 1941 throughout most of the year; whereas in the other regions the 1947 traffic in each month was well above that of the corresponding month of any previous year. The States comprising each of these regions, which have been established by the Bureau of the Census, are indicated in the first column of table 3, page 147.

Vehicle-mileage of travel on all rural roads, by type of vehicle, is shown in chart form for each year from 1936 to 1947, inclusive, in figure 2. Evident from this chart is the stability of the truck traffic. Even during the war period it decreased less than 26 percent from the prewar high, while passenger-car traffic decreased over 45 percent. Truck traffic, as well as passenger-car traffic, has regained all of the wartime loss and now stands about where indicated by the prewar trend. Travel by truck combinations during the years of the war was even more stable, and maintained throughout that period approximately the same level as in 1941. Since the end of the war period, the vehicle-mileage of truck combinations has increased annually at a rapid rate.

The ratios of rural traffic volumes in 1947 to those in corresponding months of 1946 and 1941 are shown in table 1. The uniform increases in traffic in 1947 compared to 1946, in all sections of the country, are clearly shown. With the exception of the Middle Atlantic region (in March and in December) the traffic in every region for every month was equal to, or greater than, that in the same period of the previous year. On the other hand, the com-

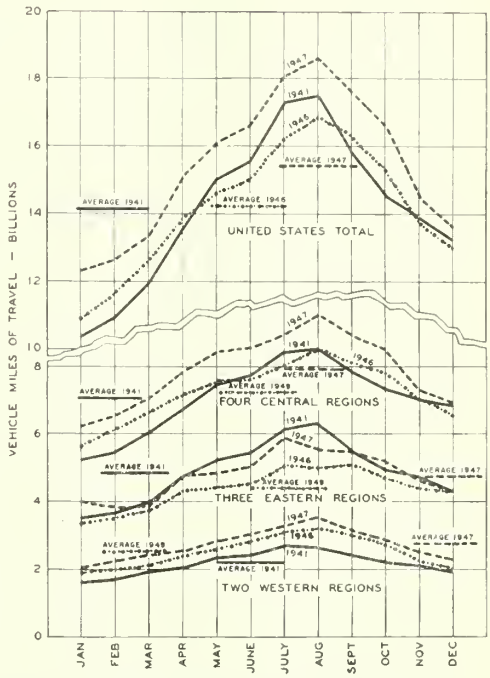


Figure 1.—Travel on all rural roads in 1941, 1946, and 1947, by months.

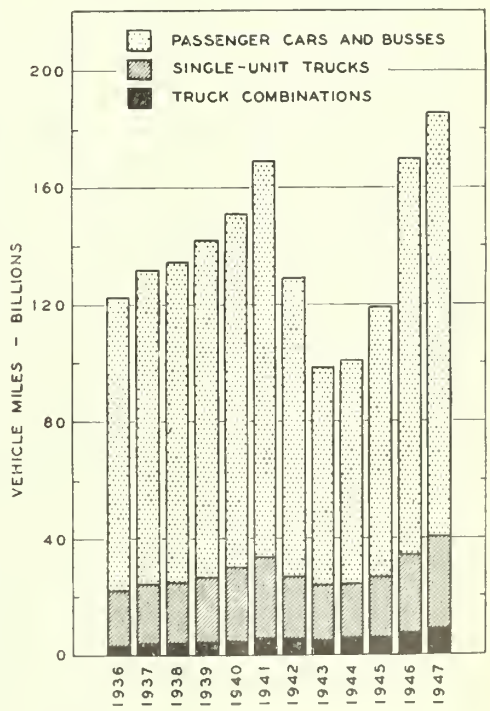


Figure 2.—Travel on all rural roads, 1936–47, by classes of vehicles.

[1] In this article, the term *trucks* is used to indicate single-unit trucks; *truck combinations* to indicate tractor-truck-semitrailers (with or without full trailers) and trucks with full trailers; and *trucks and truck combinations* to indicate all of these vehicles together.

Table 1.—*Ratio of traffic volume on rural roads in 1947 to that in corresponding months of 1946 and 1941, as determined from automatic traffic recorder data*

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
RATIO OF 1947 TRAFFIC TO 1946 TRAFFIC													
Eastern regions:													
New England	1.18	1.19	1.07	1.08	1.11	1.09	1.12	1.15	1.09	1.12	1.07	1.01	1.10
Middle Atlantic	1.17	1.11	.97	1.10	1.09	1.12	1.17	1.11	1.08	1.12	1.06	.98	1.09
South Atlantic	1.21	1.10	1.07	1.15	1.13	1.14	1.15	1.14	1.10	1.12	1.08	1.07	1.12
Average	1.19	1.11	1.04	1.12	1.11	1.12	1.15	1.13	1.09	1.12	1.07	1.03	1.11
Central regions:													
East North Central	1.21	1.10	1.09	1.12	1.14	1.15	1.13	1.12	1.11	1.11	1.03	1.03	1.11
East South Central	1.04	1.03	1.05	1.10	1.11	1.12	1.11	1.09	1.06	1.09	1.08	1.10	1.08
West North Central	1.16	1.09	1.06	1.07	1.11	1.12	1.12	1.14	1.12	1.10	1.02	1.04	1.10
West South Central	1.00	1.02	1.02	1.06	1.05	1.07	1.06	1.06	1.05	1.04	1.06	1.06	1.05
Average	1.12	1.06	1.06	1.09	1.11	1.12	1.11	1.11	1.09	1.09	1.04	1.05	1.09
Western regions:													
Mountain	1.09	1.07	1.11	1.08	1.11	1.11	1.11	1.12	1.14	1.14	1.08	1.11	1.11
Pacific	1.09	1.13	1.10	1.08	1.08	1.07	1.06	1.07	1.01	1.00	1.11	1.13	1.07
Average	1.09	1.11	1.10	1.08	1.09	1.08	1.08	1.09	1.05	1.04	1.10	1.12	1.08
United States average	1.13	1.09	1.06	1.10	1.11	1.11	1.12	1.11	1.08	1.09	1.06	1.06	1.09
RATIO OF 1947 TRAFFIC TO 1941 TRAFFIC													
Eastern regions:													
New England	1.07	.99	1.05	.91	.86	.83	.83	.81	.95	.97	.83	.81	.89
Middle Atlantic	1.08	1.07	.82	.96	.86	.87	.95	.83	1.00	1.04	.96	1.04	.95
South Atlantic	1.18	1.11	1.13	1.10	1.08	1.08	1.04	1.02	1.06	1.14	1.09	1.08	1.09
Average	1.13	1.08	.99	1.01	.94	.95	.96	.90	1.02	1.07	.99	1.02	.99
Central regions:													
East North Central	1.30	1.24	1.22	1.17	1.12	1.06	.98	1.07	1.10	1.14	.93	.90	1.09
East South Central	1.11	1.17	1.18	1.16	1.14	1.09	1.04	1.19	1.13	1.18	1.22	1.16	1.14
West North Central	1.15	1.06	1.05	1.05	1.02	1.07	1.05	1.08	1.13	1.13	1.00	.98	1.06
West South Central	1.17	1.24	1.22	1.27	1.28	1.22	1.24	1.23	1.20	1.26	1.22	1.15	1.22
Average	1.21	1.19	1.18	1.17	1.13	1.10	1.06	1.12	1.13	1.17	1.05	1.01	1.12
Western regions:													
Mountain	1.21	1.19	1.11	1.16	1.12	1.17	1.13	1.21	1.26	1.23	1.07	1.21	1.17
Pacific	1.31	1.31	1.31	1.28	1.23	1.34	1.30	1.37	1.33	1.33	1.24	1.21	1.30
Average	1.28	1.27	1.24	1.24	1.19	1.28	1.24	1.31	1.31	1.30	1.18	1.21	1.25
United States average	1.19	1.16	1.12	1.12	1.08	1.07	1.05	1.07	1.12	1.15	1.05	1.04	1.10

Table 2.—*Percentage of annual traffic in each month in 1947*

Region	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
Eastern regions:													
New England	5.8	6.2	7.1	7.9	8.7	8.8	10.5	10.6	10.4	9.8	7.7	6.5	100.0
Middle Atlantic	6.4	6.3	5.5	8.2	8.6	8.9	10.7	9.9	10.4	9.4	8.1	7.6	100.0
South Atlantic	7.6	7.3	7.7	8.4	8.3	8.7	9.4	9.1	8.5	8.6	8.3	8.1	100.0
Average	6.8	6.7	6.7	8.2	8.5	8.8	10.1	9.7	9.6	9.1	8.1	7.7	100.0
Central regions:													
East North Central	6.7	6.7	7.6	8.6	9.5	9.1	9.6	10.5	9.6	8.9	6.9	6.3	100.0
East South Central	6.4	6.9	7.4	8.0	8.5	8.4	9.0	10.0	9.3	9.1	8.6	8.4	100.0
West North Central	6.2	6.5	6.9	7.8	8.5	9.3	9.9	10.5	9.9	9.4	7.9	7.2	100.0
West South Central	7.0	7.4	7.8	8.3	8.4	8.7	8.9	9.0	8.7	8.7	8.6	8.5	100.0
Average	6.6	6.9	7.4	8.2	8.9	9.0	9.4	10.1	9.4	9.0	7.8	7.3	100.0
Western regions:													
Mountain	6.1	6.4	7.1	7.7	8.4	9.5	10.4	10.9	9.9	9.0	7.5	7.1	100.0
Pacific	6.5	7.0	7.4	7.9	8.6	9.2	9.9	10.5	9.6	8.7	7.7	7.0	100.0
Average	6.3	6.8	7.3	7.8	8.6	9.3	10.1	10.6	9.7	8.8	7.6	7.1	100.0
United States average	6.6	6.8	7.2	8.2	8.7	9.0	9.8	10.0	9.5	9.0	7.8	7.4	100.0

parison of 1947 traffic figures with those for 1941 shows that traffic was generally higher, month by month, in 1947 than in 1941 in the South Atlantic, Central, and Western regions, while travel in the New England and Middle Atlantic regions remained generally somewhat below the levels attained in the earlier year.

Table 2 indicates the differences in the regional annual traffic patterns for the year 1947. Normally, for the United States as a whole, the summer months of July and August have higher percentages of traffic than other months of the year. This is generally true for the year shown although, as would be expected, the variation is less marked in the South Atlantic and West South Central regions. In these regions storms and cold weather generally cut traffic less in winter than in the northern areas and also tourist and resort travel is relatively less in summer.

Table 4.—*Ratio of 1947 traffic to corresponding traffic in 1946, from summer counts*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	1.13	1.07	1.24	1.15	1.06	1.13	1.18	0.97	1.07	1.20	1.08	1.10	1.10
Foreign	1.07	1.18	.88	1.00	1.24	1.08	.90	1.31	1.13	1.04	1.05	1.04	1.07
All passenger cars	1.11	1.09	1.13	1.11	1.10	1.11	1.10	1.03	1.09	1.12	1.07	1.09	1.09
Trucks and truck combinations:													
Single-unit trucks	1.04	1.15	1.19	1.16	1.19	1.10	1.16	1.10	1.14	1.12	1.37	1.25	1.16
Truck combinations	1.18	1.25	1.18	1.21	1.32	1.24	1.15	1.28	1.26	1.10	1.12	1.12	1.21
All trucks and truck combinations	1.06	1.18	1.19	1.17	1.23	1.13	1.16	1.14	1.17	1.12	1.26	1.20	1.18
Busses	1.00	1.14	1.18	1.14	1.08	.96	1.27	.96	1.06	1.15	.94	1.01	1.08
All vehicles	1.10	1.11	1.15	1.13	1.12	1.11	1.11	1.06	1.10	1.12	1.10	1.11	1.11

1947 SUMMER SURVEY

During the summer of 1947, the highway departments of 44 States, in cooperation with the Public Roads Administration, conducted a survey to obtain the trends in the volume and composition of traffic, and weights of trucks and truck combinations on rural roads. This survey followed, in general, similar surveys conducted each summer for the past 5 years.[2]

All States participated in these surveys except Wyoming, Maryland, Virginia, and Illinois. In Wyoming, an expanded weight survey was in progress but reports from it were not available for inclusion in this analysis.

The majority of weighing stations were operated during July, August, or September although Arizona, California, and Florida completed their work in June, Minnesota conducted the operations from May through August, and Mississippi operated for the entire year.

The stations used in these surveys were selected initially to give a representative cross section of traffic on main rural roads, and were operated generally for 8 hours on a weekday, either from 6 a. m. to 2 p. m. or from 2 p. m. to 10 p. m. All traffic passing through the stations during the period was counted and classified into the following categories: local passenger cars; foreign (out-of-State) passenger cars; panel and pick-up trucks;[3] other 2-axle, 4-tired trucks; 2-axle, 6-tired trucks; 3-axle trucks; tractor-semitrailer combinations; truck and trailer or tractor-semitrailer and trailer combinations; and busses.

The survey period, number of stations operated, number of vehicles counted, and number weighed are shown for each State in table 3. A total of 961,597 vehicles were counted at all stations during the period of the survey. Trucks and truck combinations numbered 192,341 or 20 percent of the total counted. Of these, 95,220 or 49.5 percent were stopped and weighed.

Wherever traffic volume permitted, all trucks and truck combinations were stopped and weighed. Where this procedure was impracticable, a sample was obtained by weighing all of the uncommon types and omitting only vehicles of types sufficiently common so that a partial sample would be adequate to establish their characteristics. The type of vehicle, whether it was loaded or empty, and the number of axles were recorded. The axle spacing and total wheelbase length of the heavier vehicles[4] were measured.

[2] See *Traffic trends on rural roads in 1946*, by T. B. Dimmick and Mary E. Kipp, PUBLIC ROADS, vol. 25, No. 3, Mar. 1948; *Traffic trends on rural roads in 1945*, by T. B. Dimmick, PUBLIC ROADS, vol. 24, No. 10, Oct.-Nov.-Dec. 1946; and *Amount and characteristics of trucking on rural roads*, by J. T. Lynch and T. B. Dimmick, PUBLIC ROADS, vol. 23, No. 9, July-Aug.-Sept. 1943.

[3] Single units weighing less than 1½ tons.

[4] Single-unit trucks weighing 13 tons or more and truck combinations weighing 17 tons or more.

Passenger cars and busses were counted but not stopped for weighing.

TRAFFIC INCREASES

The ratios of 1947 summer traffic counts to corresponding counts in 1946 are given for the main vehicle types in table 4. The more detailed vehicle-type classification is not shown in this table because it was not made prior to 1947. Particularly noticeable in this table is the increase in trucks and truck combinations observed in every section of the country. The increase of single-unit trucks counted in the Pacific region and of truck combinations in the East North Central region is most noteworthy. In all regions except the New England and Mountain regions, truck and truck-combination traffic increased faster than passenger-car traffic. This was particularly true in the East North Central region where the passenger cars observed increased 10 percent while trucks and truck combinations increased 23 percent, and in the West South Central region where the passenger-car counts increased only 3 percent compared to an increase of 14 percent in the truck and truck-combination counts. Foreign passenger-car travel fell off surprisingly in the South Atlantic region and in the West North Central region.

Table 5 shows the percentage distribution of traffic by vehicle type in 1947. In this table the single-unit trucks are divided into the four new classification types that have been recommended by the Public Roads Administration and adopted by most of the States. The classification of trucks into these groups permits more positive identification than the use of the old "light, medium, and heavy" categories, and the groups are more homogeneous than those formerly used.

The percentage figures for the various types in table 5 indicate that the West South Central, East South Central, and Mountain regions have larger percentages of trucks than the areas with heavier travel such as New England, East North Central, Pacific, and Middle Atlantic regions. The table also shows that local conditions in some areas apparently encourage the use of certain types of vehicles, such as 2-axle, 6-tired trucks that are especially popular in the West North Central region, or the truck and trailer combinations that are used in the Pacific region to a greater extent than elsewhere in the country.

AVERAGE WEIGHTS NOW RELATIVELY STABLE

Figure 3 shows graphically the average weights of loaded and of empty single-unit trucks, of truck combinations, and of all trucks and truck combinations, in the summers of 1942 to 1947, inclusive, and in a corresponding period of a prewar year, generally 1936 or 1937. This chart shows that the weights of both loaded and empty single-unit trucks increased each year from the time of the first survey through 1945, and since then have decreased slightly. On the other hand, the weights of both loaded and empty truck combinations increased each year of the period shown. The average weights of the various types of loaded and empty trucks and truck combinations in the summer of 1947 are shown in table 6, for the different regions. This table brings out clearly the important differences that exist in the weight characteristics of the vehicles in the different groups of the new classification. It will be noted, for example, that for the United States as a whole the loaded 3-axle, single-unit trucks weighed

Table 3.—*Survey period, number of stations operated, number of vehicles counted, and number weighed in each State in the special weight survey during the summer of 1947*

Region and State	Survey period	Number of stations	Vehicles counted		Trucks and combinations weighed
			All vehicles	Trucks and truck combinations	
New England:					
Connecticut	Sept. 8–22	10	26,918	4,815	2,348
Maine	July 25–Aug. 26	10	25,360	4,138	1,512
Massachusetts	Aug. 18–29	10	28,565	4,848	2,219
New Hampshire	July 28–Aug. 1	5	13,891	1,714	548
Rhode Island	July 28–Aug. 1	5	11,466	2,096	970
Vermont	Aug. 4–8	5	8,385	580	580
Subtotal		45	114,585	18,191	8,177
Middle Atlantic:					
New Jersey	Aug. 4–Sept. 4	10	59,277	11,012	1,878
New York	Sept. 22–26	20	36,938	9,515	3,543
Pennsylvania	Aug. 6–19	10	36,425	6,432	1,633
Subtotal		40	132,640	26,959	7,054
South Atlantic:					
Delaware	Sept. 23–Oct. 2	4	11,004	3,148	447
Florida	June 9–30	10	12,021	2,170	2,112
Georgia	Aug. 4–23	10	12,635	2,951	1,804
Maryland	[1]				
North Carolina	Aug. 19–Sept. 9	12	23,867	5,546	3,069
South Carolina	Sept. 22–Oct. 7	10	15,147	3,843	1,923
Virginia	[1]				
West Virginia	Aug. 12–Sept. 5	9	10,330	2,735	1,244
Subtotal		55	85,004	20,393	10,599
Eastern regions, subtotal		140	332,229	65,543	25,830
East North Central:					
Illinois	[1]				
Indiana	Aug. 4–29	20	36,105	8,261	3,459
Michigan	July 24–Aug. 7	10	25,942	4,091	1,896
Ohio	July 29–Aug. 14	10	21,473	3,740	1,082
Wisconsin	Aug. 1–22	15	27,316	3,163	1,804
Subtotal		55	110,836	19,255	8,241
East South Central:					
Alabama	Aug. 14–Sept. 2	10	9,591	2,343	1,425
Kentucky	June 19–July 24	10	10,592	2,584	948
Mississippi	Jan. 27–Dec. 17	15	17,728	4,785	3,484
Tennessee	July 7–17	9	9,087	2,311	1,484
Subtotal		44	46,998	12,023	7,341
West North Central:					
Iowa	July 28–Aug. 8	10	11,498	1,960	1,947
Kansas	Aug. 25–Sept. 8	10	8,741	1,799	1,040
Minnesota	May 5–June 6 and July 7–Aug. 29	28	80,009	15,510	6,227
Missouri	July 28–Aug. 15	42	75,752	17,725	11,505
Nebraska	July 7–Aug. 13	20	18,226	3,844	3,679
North Dakota	Aug. 1–28	10	13,767	2,781	1,839
South Dakota	July 28–Aug. 14	9	4,305	925	872
Subtotal		129	212,298	44,544	27,109
West South Central:					
Arkansas	Aug. 4–15	10	13,611	4,276	1,340
Louisiana	July 28–Aug. 8	10	7,551	2,337	2,104
Oklahoma	Aug. 11–22	10	13,747	3,068	3,068
Texas	June 17–Aug. 22	17	23,728	5,392	2,273
Subtotal		47	58,637	15,073	8,785
Central regions, subtotal		275	428,769	90,895	51,476
Mountain:					
Arizona	June 9–20	10	8,920	1,672	661
Colorado	Aug. 15–29	10	23,397	3,866	1,051
Idaho	Aug. 4–20	13	14,016	2,587	2,069
Montana	July 30–Aug. 25	18	14,640	3,000	2,393
Nevada	July 29–Aug. 12	10	6,224	784	765
New Mexico	July 21–Aug. 7	10	9,033	1,901	1,542
Utah	July 28–Aug. 8	10	15,366	2,649	851
Wyoming	[2]				
Subtotal		81	91,596	16,459	9,332
Pacific:					
California	May 27–June 30	20	72,375	13,284	4,117
Oregon	Aug. 11–29	9	19,208	3,582	2,509
Washington	Sept. 3–16	10	17,420	2,578	1,956
Subtotal		39	109,003	19,444	8,582
Western regions, subtotal		120	200,599	35,903	17,914
United States total		535	961,597	192,341	95,220

[1] No survey made in 1947.
[2] Made an expanded survey in 1947 that is not comparable with previous counts.

Table 5.—*Percentage distribution of traffic, by vehicle types, in the summer of 1947*

Vehicle types	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	57.84	63.52	57.88	60.04	59.19	54.72	58.44	58.39	58.26	40.58	68.34	59.11	59.00
Foreign	24.68	15.09	17.22	17.45	23.00	21.82	18.07	15.87	19.92	36.33	11.50	19.75	19.08
All passenger cars	82.52	78.61	75.10	77.49	82.19	76.54	76.51	74.26	78.18	76.91	79.84	78.86	78.08
Single-unit trucks:													
Panel and pick-up	5.47	4.76	6.81	5.83	3.95	7.32	5.60	9.13	6.03	9.61	3.89	5.79	5.92
Other 2-axle, 4-tired	.53	1.54	.75	1.03	.20	.66	1.21	.63	.61	.43	1.63	1.23	.86
Other 2-axle, 6-tired	6.54	8.02	8.59	8.08	6.17	9.41	10.61	8.16	8.14	7.54	3.62	4.93	7.54
Three-axle	.34	.66	.88	.72	.66	.34	.21	.22	.40	.48	2.59	1.89	.77
All single-unit trucks	12.88	14.98	17.03	15.66	10.98	17.73	17.63	18.14	15.18	18.06	11.73	13.84	15.09
Truck combinations:													
Tractor-truck and semitrailer	3.18	5.31	6.23	5.45	5.82	4.05	4.66	6.30	5.44	3.30	4.49	4.10	5.19
Truck and trailer	.01	.03	.02	.02	.26	.....	.22	.40	.25	.62	3.04	2.23	.54
All truck combinations	3.19	5.34	6.25	5.47	6.08	4.05	4.88	6.70	5.69	3.92	7.53	6.33	5.73
All trucks and truck combinations	16.07	20.32	23.28	21.13	17.06	21.78	22.51	24.84	20.87	21.98	19.26	20.17	20.82
Busses	1.41	1.07	1.62	1.38	.75	1.68	.98	.90	.95	1.11	.90	.97	1.10
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

more than twice as much as the 2-axle, 6-tired trucks, and that similar differences existed throughout the classification.

TRUCK TRANSPORTATION HAS PHENOMENAL INCREASE

Figure 4 shows a comparison of the estimated vehicle-miles of travel on main rural roads[5] of both the loaded and the empty single-unit trucks and truck combinations, separately and combined, for each year from 1936 to 1947, inclusive. This chart shows more clearly than figure 2 the steady growth of truck traffic during the period 1936–41, the temporary effect of the wartime restrictions, and the phenomenal upsurge in highway truck transportation since the war. Travel by single-unit trucks in 1947 increased 16 percent above that for 1946; that of truck combinations increased 21 percent; and that of all types increased 18 percent. The increases of travel by loaded vehicles, considered separately from empty vehicles, were even greater. Travel by loaded single-unit trucks in 1947 increased 21 percent above that for 1946; that of truck combinations increased 23 percent; and that of all types increased 22 percent.

Table 7 gives a comparison, by regions, of the estimated vehicle-miles of travel by vehicles of different types on all main rural roads in 1947 and 1946. While total travel increased a considerable amount (11 percent) over the previous year's total, even larger increases were found in the travel by freight-carrying vehicles. These increases were indicated in every census region but were more noticeable in the Pacific region, where truck and truck-combination traffic increased 26 percent over the previous total, and in the East North Central region where it increased 23 percent above the 1946 estimate.

A second important trend that is indicated by the figures in table 7 is the continued increase found in many sections of the country in the more general use of the heavier commercial vehicles. In every region east of the Rocky Mountain States, except in the West North Central and the South Atlantic regions, the vehicle-mileage of truck combinations increased more rapidly than that of single-unit trucks. In the Pacific region, on the other hand, where the use of loaded truck combinations had increased phenomenally in previous years, the vehicle-mileage of this type increased only 12 percent as compared to 37 percent for single-unit trucks.

Figure 5 gives a comparison of the average load carried by single-unit trucks, truck combinations, and all of these vehicles combined, in the 12 years since the planning surveys were commenced in 1936. The chart shows the general upward trend of load weights throughout all except the last 2 years of this period. In 1947 the average carried load both of single-unit trucks and of truck combinations declined slightly from the 1946 level but, because of the increased proportion of combinations, the average for all vehicles remained practically constant. From 1936 to 1947 the average carried load on single-unit trucks increased from 1.9 tons to 2.3 tons, an increase of 21 percent; the average

[5] Main rural roads, as referred to in this article, comprise about 345,000 miles of the principal routes in the United States.

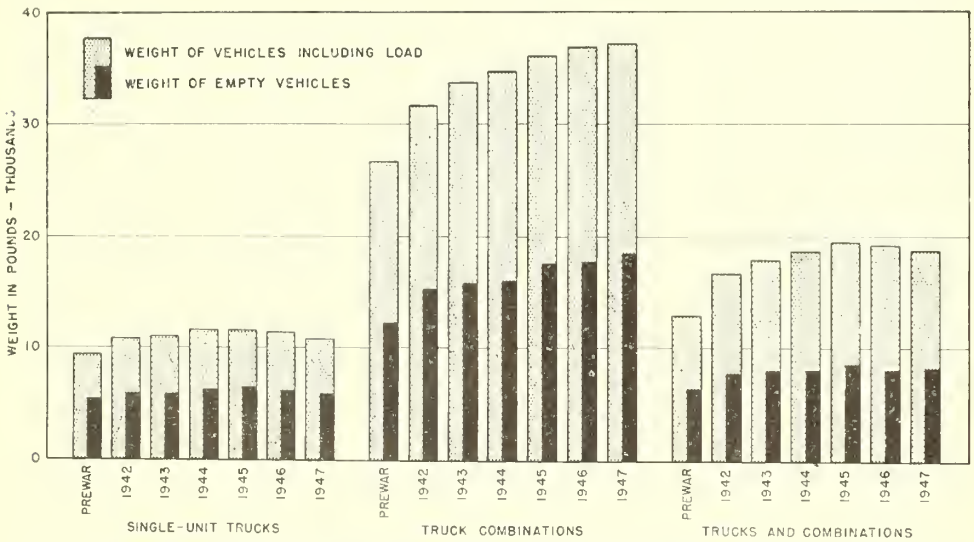


Figure 3.—Average weights of loaded and of empty trucks and truck combinations in the summers of 1942–47 and in a corresponding period of a prewar year.

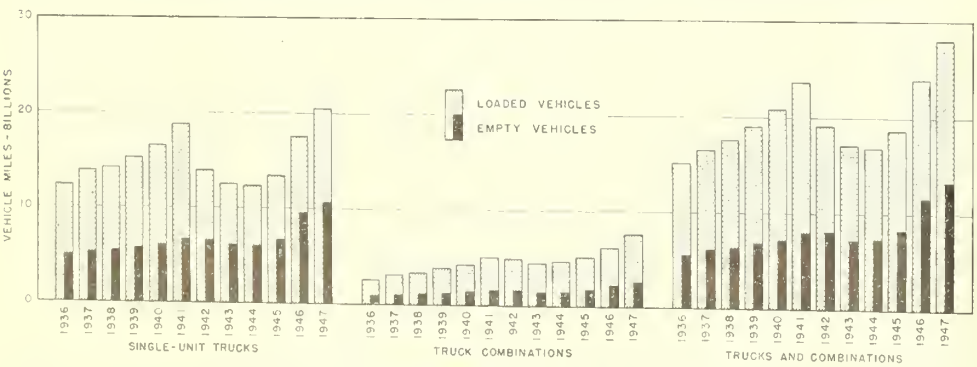


Figure 4.—Travel on main rural roads by loaded and empty trucks and truck combinations.

Table 6.—*Average weights (in pounds) of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1947*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
AVERAGE WEIGHTS OF LOADED TRUCKS													
Single-unit trucks:													
Panel and pick-up	6,321	4,774	5,127	5,187	5,462	4,990	4,800	5,378	5,199	5,681	4,070	4,660	5,109
Other 2-axle, 4-tired	9,320	7,044	8,770	7,872	8,120	6,207	6,738	7,897	7,227	8,580	8,258	8,277	7,816
Other 2-axle, 6-tired	14,544	14,216	12,221	13,227	13,485	12,685	12,420	12,452	12,805	13,193	11,246	12,083	12,871
Three-axle	29,355	28,122	24,104	25,595	32,809	28,920	24,115	24,308	30,676	24,837	25,169	25,145	26,711
Average	11,779	11,786	10,247	10,978	12,314	10,963	10,127	9,038	10,622	10,776	11,774	11,507	10,897
Truck combinations:													
Tractor-truck and semitrailer	37,141	39,358	34,726	36,557	35,422	30,704	36,711	32,122	34,371	40,843	45,179	44,059	36,594
Truck and trailer	(1)	(1)	(1)	(1)	57,614		19,081	19,704	33,259	51,630	52,378	52,099	46,574
Average	37,223	39,293	34,713	36,560	36,405	30,704	35,600	31,197	34,312	42,215	47,771	46,625	37,498
Average, all trucks and truck combinations	18,196	20,341	16,792	18,214	23,663	16,396	16,869	15,639	18,862	19,427	22,028	21,111	19,047
AVERAGE WEIGHTS OF EMPTY TRUCKS													
Single-unit trucks:													
Panel and pick-up	4,689	3,976	3,744	3,937	4,109	3,943	3,730	4,273	4,057	3,967	3,468	3,777	3,965
Other 2-axle, 4-tired	7,103	4,992	5,740	5,327	6,110	4,680	4,718	5,339	5,060	5,221	5,545	5,485	5,271
Other 2-axle, 6-tired	8,412	6,137	6,898	6,779	6,991	7,172	7,160	7,189	7,129	7,184	7,514	7,328	7,036
Three-axle	16,899	13,339	11,054	12,480	15,048	11,835	12,248	12,099	13,766	13,514	14,605	14,515	13,777
Average	6,719	6,687	5,456	6,073	6,046	5,502	5,730	5,579	5,716	5,244	7,324	6,267	5,932
Truck combinations:													
Tractor-truck and semitrailer	20,499	19,605	17,075	18,399	18,344	15,813	17,765	15,830	17,094	20,429	21,100	20,814	17,942
Truck and trailer	(1)	(1)	(1)	(1)	24,391		11,609	15,773	20,130	28,252	21,930	22,982	21,933
Average	20,498	19,623	17,072	18,405	18,871	15,813	17,528	15,816	17,249	21,778	21,531	21,606	18,304
Average, all trucks and truck combinations	8,536	9,110	7,484	8,232	9,854	6,555	7,834	8,017	8,153	7,228	10,520	8,959	8,321

[1] Data omitted because of insufficient sample.

Table 7.—*Comparison of estimated 1947 vehicle-miles of travel on main rural roads with corresponding figures for 1946*

Region and year	All vehicles, vehicle-miles	Passenger cars and busses [1]		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
Eastern regions:									
New England:	*Thousands*		*Thousands*		*Thousands*		*Thousands*		*Thousands*
1946	5,713,086	83.3	4,759,184	16.7	953,902	82.1	783,031	17.9	170,871
1947	6,298,391	83.9	5,286,129	16.1	1,012,262	80.1	811,285	19.9	200,977
1947: 1946 ratio	1.10	1.01	1.11	.96	1.06	.98	1.04	1.11	1.18
Middle Atlantic:									
1946	15,532,600	80.9	12,564,265	19.1	2,968,335	75.3	2,234,986	24.7	733,349
1947	17,207,662	79.7	13,711,322	20.3	3,496,340	73.7	2,577,225	26.3	919,115
1947: 1946 ratio	1.11	.99	1.09	1.06	1.18	.98	1.15	1.06	1.25
South Atlantic:									
1946	18,439,919	77.5	14,290,746	22.5	4,149,173	72.9	3,025,426	27.1	1,123,747
1947	21,193,968	76.7	16,260,267	23.3	4,933,701	73.2	3,609,723	26.8	1,323,978
1947: 1946 ratio	1.15	.99	1.14	1.04	1.19	1.00	1.19	.99	1.18
Subtotal:									
1946	39,685,605	79.7	31,614,195	20.3	8,071,410	74.9	6,043,443	25.1	2,027,967
1947	44,700,021	78.9	35,257,718	21.1	9,442,303	74.1	6,998,233	25.9	2,444,070
1947: 1946 ratio	1.13	.99	1.12	1.04	1.17	.99	1.16	1.03	1.21
Central regions:									
East North Central:									
1946	23,563,034	84.4	19,898,197	15.6	3,664,837	66.7	2,444,116	33.3	1,220,721
1947	26,432,337	82.9	21,923,580	17.1	4,508,757	64.4	2,902,341	35.6	1,606,416
1947: 1946 ratio	1.12	.98	1.10	1.10	1.23	.97	1.19	1.07	1.32
East South Central:									
1946	7,470,206	78.5	5,867,331	21.5	1,602,875	83.1	1,332,105	16.9	270,770
1947	8,288,771	78.2	6,482,965	21.8	1,805,806	81.4	1,469,955	18.6	335,851
1947: 1946 ratio	1.11	.99	1.10	1.01	1.13	.98	1.10	1.10	1.24
West North Central:									
1946	14,971,991	78.5	11,755,049	21.5	3,216,942	78.2	2,514,597	21.8	702,345
1947	16,528,990	77.5	12,807,372	22.5	3,721,618	78.3	2,914,132	21.7	807,486
1947: 1946 ratio	1.10	.99	1.09	1.05	1.16	1.00	1.16	1.00	1.15
West South Central:									
1946	15,721,893	77.2	12,130,839	22.8	3,591,054	75.9	2,724,193	24.1	866,861
1947	16,533,940	75.2	12,426,948	24.8	4,106,992	73.0	2,998,788	27.0	1,108,204
1947: 1946 ratio	1.05	.97	1.02	1.09	1.14	.96	1.10	1.12	1.28
Subtotal:									
1946	61,727,124	80.4	49,651,416	19.6	12,075,708	74.7	9,015,011	25.3	3,060,697
1947	67,784,038	79.1	53,640,865	20.9	14,143,173	72.7	10,285,216	27.3	3,857,957
1947: 1946 ratio	1.10	.98	1.08	1.07	1.17	.97	1.14	1.08	1.26
Western regions:									
Mountain:									
1946	7,476,867	78.1	5,837,284	21.9	1,639,583	82.0	1,343,755	18.0	295,828
1947	8,324,525	78.0	6,495,071	22.0	1,829,454	82.2	1,503,184	17.8	326,270
1947: 1946 ratio	1.11	1.00	1.11	1.00	1.12	1.00	1.12	.99	1.10
Pacific:									
1946	15,259,494	83.2	12,700,388	16.8	2,559,106	56.1	1,435,685	43.9	1,123,421
1947	16,703,515	80.7	13,486,242	19.3	3,217,273	60.9	1,959,729	39.1	1,257,544
1947: 1946 ratio	1.09	.97	1.06	1.15	1.26	1.09	1.37	.89	1.12
Subtotal:									
1946	22,736,361	81.5	18,537,672	18.5	4,198,689	66.2	2,779,440	33.8	1,419,249
1947	25,028,040	79.8	19,981,313	20.2	5,046,727	68.6	3,462,913	31.4	1,583,814
1947: 1946 ratio	1.10	.98	1.08	1.09	1.20	1.04	1.25	.93	1.12
United States total:									
1946	124,149,090	80.4	99,803,283	19.6	24,345,807	73.3	17,837,894	26.7	6,507,913
1947	137,512,099	79.2	108,879,896	20.8	28,632,203	72.5	20,746,362	27.5	7,885,841
1947: 1946 ratio	1.11	.99	1.09	1.06	1.18	.99	1.16	1.03	1.21

[1] Percentages of total 1947 travel by passenger cars and by busses is reported separately in table 5.

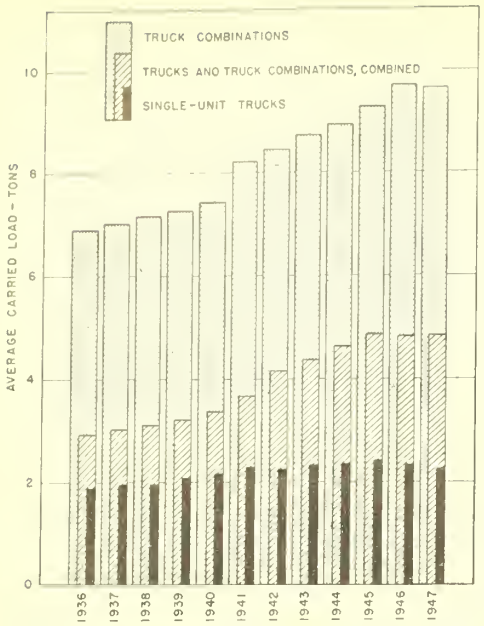


Figure 5.—Average load carried by trucks and truck combinations on main rural roads, 1936–47.

load carried by truck combinations increased from 6.9 tons to 9.6 tons, or 39 percent; and the average of all types combined increased from 2.9 tons to 4.8 tons, or 66 percent.

Figure 6 shows a comparison, for each year from 1936 through 1947, of the ton-miles of freight carried by trucks and truck combinations on main rural roads. The chart demonstrates how truck combinations are transporting each year a larger portion of the total

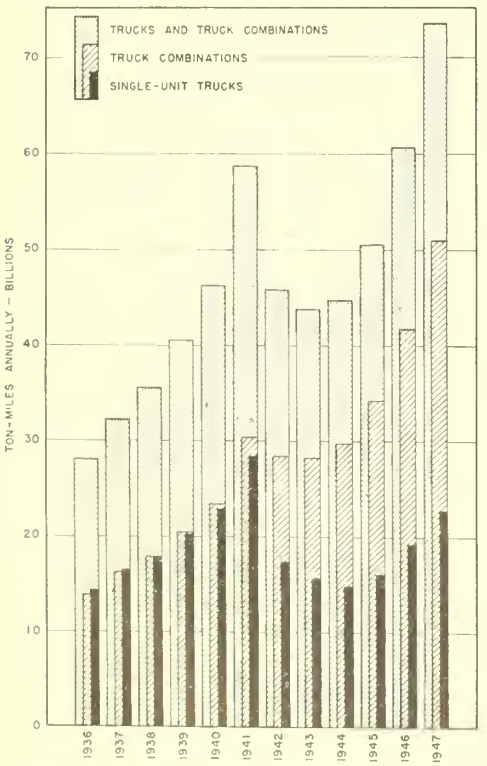


Figure 6.—Estimated ton-miles carried by trucks and truck combinations on main rural roads, 1936–47.

amount of highway freight. In 1936 the truck combinations accounted for approximately the same ton-mileage as the single-unit trucks while in 1947 the ratio increased to more than two to one. The sharp increase in the total ton-mileage during the last 3 years is especially striking.

GREATER PERCENTAGE OF LOADED TRUCKS

In table 8 is shown a comparison of the percentage of vehicles carrying loads, the average carried load, and the ton-mileage carried for all trucks and combinations, for single-unit trucks, and for truck combinations in 1947 and in 1946, by census regions. For the country as a whole, from 1946 to 1947 the percentage of trucks and truck combinations carrying loads increased from 51.7 percent to 53.5 percent, that of single-unit trucks increased from 46.4 percent to 48.3 percent, while that of truck combinations increased from 66.2 percent to 67.1 percent. These increases are particularly interesting because they reverse a trend of decreasing percentages of loaded vehicles that had prevailed since the beginning of the war, due probably to the use of trucks as passenger cars during the period of gasoline rationing and vehicle shortage. It is to be noted than even in 1947, over half of the single-unit trucks were empty, which may reflect the continuing shortage of new automobiles.

As pointed out in the discussion of figure 5, the average carried loads for the country as a whole were about the same in 1947 as in 1946. Table 8 shows that there was no great change in any of the regions, the greatest increase being 7 percent in the Middle Atlantic region and the greatest decrease being 9 percent in the Pacific region.

The ton-mileage transported by all types of freight-carrying vehicles, traveling on main rural roads, increased about 21 percent in 1947 compared to 1946; that by single-unit trucks increasing 18 percent, and that by truck combinations increasing 22 percent. Increases of the 1947 estimates over those for 1946 were found in every region both for single-unit trucks and for truck combinations. The percentage increases for trucks and truck combinations combined ranged from 9 percent in the

Table 8.—*Comparison of the estimated 1947 percentage of trucks and truck combinations loaded, average carried load, and ton-miles carried on main rural roads, with corresponding figures for 1946*

Region and year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
Eastern regions:		*Tons*	*Thousands*		*Tons*	*Thousands*		*Tons*	*Thousands*
New England:									
1946	52.4	3.99	1,995,842	48.2	2.24	844,357	57.2	9.40	1,151,485
1947	54.1	3.99	2,187,043	50.5	2.10	860,482	69.0	9.56	1,326,561
1947: 1946 ratio	1.03	1.00	1.10	1.05	.94	1.02	1.21	1.02	1.15
Middle Atlantic:									
1946	57.7	4.10	7,023,600	55.8	2.18	2,719,736	63.4	9.25	4,303,864
1947	52.1	4.40	8,025,404	48.9	2.35	2,965,474	61.2	8.99	5,059,930
1947: 1946 ratio	.90	1.07	1.14	.88	1.08	1.09	.97	.97	1.18
South Atlantic:									
1946	53.5	4.79	10,637,066	48.4	2.38	3,494,145	67.0	9.48	7,142,921
1947	55.7	4.55	12,505,330	51.6	2.16	4,021,377	66.8	9.58	8,483,953
1947: 1946 ratio	1.04	.95	1.18	1.07	.91	1.15	1.00	1.01	1.19
Subtotal:									
1946	54.9	4.44	19,656,508	51.1	2.29	7,058,238	66.1	9.39	12,598,270
1947	54.2	4.44	22,717,777	50.4	2.22	7,847,333	64.9	9.37	14,870,444
1947: 1946 ratio	.99	1.00	1.16	.99	.97	1.11	.98	1.00	1.18
Central regions:									
East North Central:									
1946	57.2	4.81	10,081,218	50.5	2.17	2,685,088	70.6	8.59	7,396,130
1947	59.1	4.92	13,116,456	52.8	2.18	3,337,759	70.5	8.63	9,778,697
1947: 1946 ratio	1.03	1.02	1.30	1.05	1.00	1.24	1.00	1.00	1.32
East South Central:									
1946	42.7	4.04	2,761,538	38.6	2.54	1,304,071	62.6	8.60	1,457,467
1947	44.6	4.21	3,384,394	39.4	2.52	1,455,767	67.3	8.54	1,928,627
1947: 1946 ratio	1.04	1.04	1.23	1.02	.99	1.12	1.08	.99	1.32
West North Central:									
1946	55.1	3.72	6,585,211	52.2	1.97	2,582,219	65.2	8.74	4,002,992
1947	55.9	3.77	7,831,725	53.0	1.93	2,984,823	66.3	9.05	4,846,902
1947: 1946 ratio	1.01	1.01	1.19	1.02	.98	1.16	1.02	1.04	1.21
West South Central:									
1946	36.6	4.16	5,461,533	31.3	2.65	2,264,895	53.2	6.93	3,196,638
1947	46.1	4.14	7,832,121	41.2	2.51	3,105,980	59.3	7.19	4,726,141
1947: 1946 ratio	1.26	.99	1.43	1.32	.95	1.37	1.11	1.04	1.48
Subtotal:									
1946	48.6	4.24	24,889,500	43.4	2.26	8,836,273	63.7	8.23	16,053,227
1947	52.6	4.32	32,164,696	47.6	2.22	10,884,329	66.1	8.34	21,280,367
1947: 1946 ratio	1.08	1.02	1.29	1.10	.98	1.23	1.04	1.01	1.33
Western regions:									
Mountain:									
1946	43.3	4.88	3,463,639	38.1	2.59	1,327,577	66.8	10.81	2,136,062
1947	39.8	5.16	3,763,238	34.5	2.61	1,354,717	64.4	11.48	2,408,521
1947: 1946 ratio	.92	1.06	1.09	.91	1.01	1.02	.96	1.06	1.13
Pacific:									
1946	62.0	8.12	12,882,143	53.3	2.45	1,878,801	73.1	13.40	11,003,342
1947	62.8	7.40	14,964,484	54.9	2.35	2,523,948	75.3	13.14	12,440,536
1947: 1946 ratio	1.01	.91	1.16	1.03	.96	1.34	1.03	.98	1.13
Subtotal:									
1946	54.7	7.12	16,345,782	46.0	2.51	3,206,378	71.8	12.90	13,139,404
1947	54.5	6.81	18,727,722	46.0	2.43	3,878,665	73.0	12.84	14,849,057
1947: 1946 ratio	1.00	.96	1.15	1.00	.97	1.21	1.02	1.00	1.13
United States total:									
1946	51.7	4.84	60,891,790	46.4	2.31	19,100,889	66.2	9.70	41,790,901
1947	53.5	4.81	73,610,195	48.3	2.26	22,610,327	67.1	9.63	50,999,868
1947: 1946 ratio	1.03	.99	1.21	1.04	.98	1.18	1.01	.99	1.22

Table 9.—*Percentage of vehicle-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried by various types of trucks and truck combinations on main rural roads in the summer of 1947*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
PERCENTAGE OF VEHICLE-MILES OF TRAVEL													
Single-unit trucks:													
Panel and pick-up	34.02	23.42	29.25	27.60	23.15	33.59	24.88	36.77	28.89	43.73	20.19	28.72	28.44
Other 2-axle, 4-tired	3.30	7.60	3.23	4.86	1.19	3.03	5.36	2.51	2.91	1.96	8.49	6.12	4.12
Other 2-axle, 6-tired	40.68	39.46	36.91	38.26	36.19	43.19	47.14	32.85	38.99	34.32	18.80	27.43	36.18
Three-axle	2.15	3.23	3.77	3.40	3.84	1.59	.92	.89	1.93	2.16	13.43	9.35	3.72
All single-unit trucks	80.15	73.71	73.16	74.12	64.37	81.40	78.30	73.02	72.72	82.17	60.91	68.62	72.46
Truck combinations:													
Tractor-truck and semitrailer	19.76	26.15	26.77	25.79	34.12	18.60	20.73	25.36	26.07	15.01	23.33	20.31	24.96
Truck and trailer	.09	.14	.07	.09	1.51	------	.97	1.62	1.21	2.82	15.76	11.07	2.58
All truck combinations	19.85	26.29	26.84	25.88	35.63	18.60	21.70	26.98	27.28	17.83	39.09	31.38	27.54
All trucks and truck combinations	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
PERCENTAGE LOADED													
Single-unit trucks:													
Panel and pick-up	44.6	41.6	42.1	42.3	42.6	25.9	44.9	38.8	39.2	23.7	45.7	33.5	39.2
Other 2-axle, 4-tired	53.8	38.7	49.2	43.4	61.8	39.0	50.6	46.9	49.6	36.3	67.3	63.7	50.9
Other 2-axle, 6-tired	54.9	55.3	58.0	56.6	58.1	49.3	57.5	43.7	53.1	47.4	60.0	53.6	54.4
Three-axle	54.7	47.6	64.2	57.7	62.1	54.8	56.0	34.6	56.9	48.9	53.6	53.2	55.5
Average	50.5	48.9	51.6	50.4	52.8	39.4	53.0	41.2	47.6	34.5	54.9	46.0	48.3
Truck combinations:													
Tractor-truck and semitrailer	68.9	61.1	66.7	64.8	71.2	67.3	65.9	58.2	66.1	65.6	80.7	76.7	67.2
Truck and trailer	88.6	86.2	72.8	81.5	54.9	------	75.2	76.2	67.5	57.4	67.3	66.3	66.8
Average	69.0	61.2	66.8	64.9	70.5	67.3	66.3	59.3	66.1	64.4	75.3	73.0	67.1
Average, all trucks and truck combinations	54.1	52.1	55.7	54.2	59.1	44.6	55.9	46.1	52.6	39.8	62.8	54.5	53.5
AVERAGE CARRIED LOAD, IN TONS													
Single-unit trucks:													
Panel and pick-up	.73	.42	.64	.59	.61	.54	.52	.82	.66	.96	.46	.65	.63
Other 2-axle, 4-tired	1.43	.93	1.45	1.18	.92	.86	1.02	1.79	1.17	1.94	1.51	1.54	1.29
Other 2-axle, 6-tired	2.87	3.01	2.69	2.83	2.66	3.12	2.55	4.13	2.98	3.41	2.39	2.85	2.91
Three-axle	6.31	8.05	5.66	6.39	4.79	9.77	5.15	8.53	5.65	6.46	5.38	5.47	5.80
Average	2.10	2.35	2.16	2.22	2.18	2.52	1.93	2.51	2.22	2.61	2.35	2.43	2.26
Truck combinations:													
Tractor-truck and semitrailer	9.50	9.00	9.59	9.36	8.36	8.54	9.36	7.50	8.37	10.87	11.97	11.72	9.25
Truck and trailer	18.99	8.66	7.81	9.53	16.60	------	3.21	3.61	7.72	15.15	15.22	15.22	13.38
Average	9.56	8.99	9.58	9.37	8.63	8.54	9.05	7.19	8.34	11.48	13.14	12.84	9.63
Average, all trucks and truck combinations	3.99	4.40	4.55	4.44	4.92	4.21	3.77	4.14	4.32	5.16	7.40	6.81	4.81
PERCENTAGE OF TON-MILEAGE CARRIED													
Single-unit trucks:													
Panel and pick-up	5.12	1.79	3.11	2.84	2.08	2.51	2.74	6.16	3.28	4.82	.90	1.69	2.74
Other 2-axle, 4-tired	1.18	1.19	.91	1.03	.23	.54	1.32	1.10	.74	.67	1.86	1.62	1.06
Other 2-axle, 6-tired	29.62	28.58	22.73	25.46	19.21	35.41	32.79	31.03	27.10	27.10	5.79	10.07	22.26
Three-axle	3.42	5.39	5.41	5.21	3.93	4.55	1.26	1.37	2.72	3.41	8.32	7.33	4.66
All single-unit trucks	39.34	36.95	32.16	34.54	25.45	43.01	38.11	39.66	33.84	36.00	16.87	20.71	30.72
Truck combinations:													
Tractor-truck and semitrailer	59.95	62.60	67.69	65.15	69.83	56.99	60.78	58.00	63.40	52.07	48.44	49.17	60.32
Truck and trailer	.71	.45	.15	.31	4.72	------	1.11	2.34	2.76	11.93	34.69	30.12	8.96
All truck combinations	60.66	63.05	67.84	65.46	74.55	56.99	61.89	60.34	66.16	64.00	83.13	79.29	69.28
All trucks and truck combinations	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Mountain region to 43 percent in the West South Central region.

Table 9 gives detailed information concerning the percentage distribution of vehicle-miles of travel, the percentage of vehicles carrying loads, the average carried load, and the percentage distribution of ton-miles transported by each type of vehicle in each of the census regions. Many interesting and useful comparisons can be made from this table. For instance, while panel and pick-up trucks in the New England region travel 34 percent of the vehicle-miles, they account for only 5 percent of the ton-mileage; in the same region the tractor-semitrailer combinations travel less than 20 percent of the vehicle-miles but account for almost 60 percent of the ton-mileage.

From the portion of table 9 showing the percentage of vehicles carrying loads, by types, it can be observed that, for the country as a whole, the panel and pick-up trucks are found empty over 60 percent of the time, probably indicating that they are being used frequently for the transportation of persons in lieu of passenger cars. The percentage of vehicles carrying loads increases directly as the size of the vehicle type, and the truck-combination type is loaded 67 percent of the time. There are variations from the general trend in some regions because of local conditions, as would be expected.

The detailed data concerning the average carried load for the various vehicle types show that carried loads increase regularly as the size of the vehicle-type increases except that, in some regions, average carried load for the truck and trailer combinations is low, probably due to the prevalence of light home-made types of trailers.

The percentages of ton-mileage carried by vehicle types are particularly useful in judging the relative importance of any portion of the freight-carrying traffic. The table shows that the bulk of all freight movement in each area is hauled in the heavier vehicles. As has been pointed out previously, the data collected in past surveys indicate that travel by the heavier vehicles is steadily increasing, a fact that will make a comparison of future tables of this type with that for 1947 of great importance.

Figure 7 shows, for the United States as a whole, the frequency of gross weights of 30,000 pounds or more, of 40,000 pounds or more, and of 50,000 pounds or more. These heavy weights are much more frequent than in the prewar (1936–41) period. The frequency in the heaviest category did not increase from 1946 to 1947, however, and those in the other two categories declined somewhat, indicating that the upward trend in gross weight frequencies may have been halted, at least temporarily.

The gross weight data by vehicle type and region are presented in table 10. Heavy gross

Table 10.—*Number of heavy gross weights per 1,000 loaded and empty trucks and truck combinations of various types on main rural roads in the summer of 1947 and of all types in the corresponding period of 1946*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 30,000 POUNDS OR MORE													
Single-unit trucks:													
Panel and pick-up	0	0	0	0	0	0	0	0	0	0	0	0	0
Other 2-axle, 4-tired	3	0	0	(1)	0	0	0	0	0	0	0	0	(1)
Other 2-axle, 6-tired	22	16	1	9	0	0	1	(1)	(1)	1	1	1	3
Three-axle	251	181	102	140	230	164	96	80	187	123	186	181	171
Average	18	12	4	9	4	3	2	1	2	4	21	14	6
Truck combinations:													
Tractor-truck and semitrailer	515	432	427	436	480	426	484	332	432	475	578	548	449
Truck and trailer	689	99	0	121	651	0	2	110	413	723	712	713	617
Average	516	430	427	436	490	426	464	319	431	519	628	603	464
Average, all trucks and truck combinations, 1947	127	116	95	107	154	79	123	87	116	90	197	158	120
Average, all trucks and truck combinations, 1946	118	149	125	133	194	121	112	65	124	92	193	154	132
NUMBER PER 1,000 WEIGHING 40,000 POUNDS OR MORE													
Single-unit trucks:													
Panel and pick-up	0	0	0	0	0	0	0	0	0	0	0	0	0
Other 2-axle, 4-tired	0	0	0	0	0	0	0	0	0	0	0	0	0
Other 2-axle, 6-tired	1	1	0	(1)	0	0	0	0	0	0	(1)	(1)	(1)
Three-axle	115	31	0	19	10	32	9	35	15	24	28	27	22
Average	3	2	0	1	(1)	1	(1)	(1)	(1)	1	3	2	1
Truck combinations:													
Tractor-truck and semitrailer	285	215	173	198	227	132	236	114	187	273	433	385	217
Truck and trailer	689	0	0	67	465	0	0	75	236	436	453	451	386
Average	292	213	173	198	241	132	224	112	189	298	441	408	232
Average, all trucks and truck combinations, 1947	64	55	39	48	76	25	55	31	50	52	120	95	57
Average, all trucks and truck combinations, 1946	53	80	44	58	80	41	45	18	47	53	133	102	60
NUMBER PER 1,000 WEIGHING 50,000 POUNDS OR MORE													
Single-unit trucks:													
Panel and pick-up	0	0	0	0	0	0	0	0	0	0	0	0	0
Other 2-axle, 4-tired	0	0	0	0	0	0	0	0	0	0	0	0	0
Other 2-axle, 6-tired	0	0	0	0	0	0	0	0	0	0	0	0	0
Three-axle	11	0	0	1	10	0	9	18	10	0	0	0	3
Average	(1)	0	0	(1)	(1)	0	(1)	(1)	(1)	0	0	0	(1)
Truck combinations:													
Tractor-truck and semitrailer	74	58	34	46	88	14	93	26	64	150	314	265	84
Truck and trailer	689	0	0	67	356	0	0	75	182	321	371	366	163
Average	82	57	34	47	104	14	88	29	70	180	337	301	92
Average, all trucks and truck combinations, 1947	17	14	8	11	45	3	20	8	22	31	87	67	26
Average, all trucks and truck combinations, 1946	9	25	6	13	36	4	14	6	17	31	104	75	26

[1] Less than 5 in 10,000.

weights are much more frequent in the Pacific region than in other parts of the country: 87 out of each 1,000 trucks and truck combinations on the main rural highways, empties included, weighed 50,000 pounds or more in 1947. This, however, is a drop from the 1946 figure of 104 for each 1,000 in this heavy category. These vehicles are entirely of the truck-combination type; approximately one-third of all combinations observed, or approximately one-half of the loaded ones, weighing 50,000 pounds or more. In the South Atlantic and the two South Central regions, these heavy gross weights were less than one-tenth as frequent as in the Pacific region.

HEAVY AXLE LOADS MORE FREQUENT

From figure 8, the rapid increase in heavy axle-load frequencies since the prewar (1936–41) period is apparent. Axle loads in excess of 22,000 pounds increased in frequency year by year throughout the period, rising from 2 per 1,000 vehicles in the prewar years to 14 per 1,000 vehicles in 1947, an increase of almost 600 percent. Somewhat lesser increases occurred in the frequencies of axle loads from 18,000 to 22,000 pounds.

Since the increases in axle-load frequencies are of the greatest significance in road and bridge design, it is worth while to examine the data by vehicle type and region, as presented in table 11. Though the greatest frequency of heavy gross weights is in the Pacific region, as previously shown, the lowest frequency of heavy axle loads is in that same region. The greatest frequency is in the Middle Atlantic region, and the next greatest in the New England region. Further, the frequency of axle loads of 22,000 pounds or more increased from 1946 to 1947 in each region except the Pacific and Mountain regions, where the comparatively low values of 1946 continued to prevail.

The relative infrequency of heavy axle loads in the Pacific region in the presence of a high frequency of heavy gross weights undoubtedly indicates a better distribution of load over a larger number of axles, on the average. In the Middle Atlantic and New England regions, however, it is the tractor-truck-semitrailer combinations rather than single-unit trucks which account for most of the heavy axle loads. This may be due to differences in the design of the prevalent types of combinations, to differences in State laws, or to differences in enforcement practices.

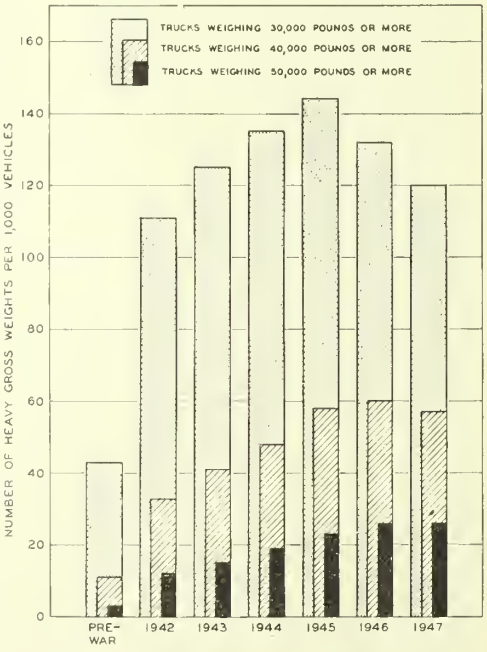


Figure 7.—Number of heavy gross weights per 1,000 trucks and truck combinations (empties included) in the summers of 1942–47 and a prewar year.

Table 11.—*Number of heavy axle loads per 1,000 loaded and empty trucks and truck combinations of various types on main rural roads in the summer of 1947 and of all types in 1946*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 18,000 POUNDS OR MORE													
Single-unit trucks:													
Panel and pick-up	0	1	0	(1)	0	3	0	0	(1)	0	0	0	(1)
Other 2-axle, 4-tired	10	0	0	1	0	0	0	0	0	0	0	0	(1)
Other 2-axle, 6-tired	51	47	13	30	8	22	5	6	9	15	15	15	17
Three-axle	105	46	15	32	65	37	23	38	53	32	33	33	38
Average	28	32	7	19	6	14	3	3	5	7	15	12	7
Truck combinations:													
Tractor-truck and semitrailer	401	510	282	380	223	200	226	152	200	189	145	158	258
Truck and trailer	0	100	0	55	473	0	2	123	252	77	68	69	116
Average	401	508	281	377	237	200	214	150	203	169	114	127	245
Average, all trucks and truck combinations, 1947	126	152	74	109	103	54	57	43	66	35	44	41	76
Average, all trucks and truck combinations, 1946	88	151	80	107	93	39	49	13	50	46	42	44	68
NUMBER PER 1,000 WEIGHING 20,000 POUNDS OR MORE													
Single-unit trucks:													
Panel and pick-up	0	0	0	0	0	0	0	0	0	0	0	0	0
Other 2-axle, 4-tired	7	0	0	(1)	0	0	0	0	0	0	0	0	(1)
Other 2-axle, 6-tired	33	28	5	17	3	6	1	0	2	7	4	5	8
Three-axle	44	25	10	18	0	12	9	19	5	0	3	3	7
Average	18	19	3	9	2	3	1	(1)	1	3	3	3	4
Truck combinations:													
Tractor-truck and semitrailer	246	316	120	205	77	79	73	56	70	75	41	51	115
Truck and trailer	0	0	0	0	183	0	2	63	102	13	4	5	34
Average	246	314	120	204	83	79	69	56	72	64	26	35	108
Average, all trucks and truck combinations, 1947	74	93	32	59	36	22	18	15	23	13	9	11	33
Average, all trucks and truck combinations, 1946	50	85	27	51	27	16	13	4	15	16	6	10	26
NUMBER PER 1,000 WEIGHING 22,000 POUNDS OR MORE													
Single-unit trucks:													
Panel and pick-up	0	0	0	0	0	0	0	0	0	0	0	0	0
Other 2-axle, 4-tired	7	0	0	1	0	0	0	0	0	0	0	0	(1)
Other 2-axle, 6-tired	19	14	2	9	0	1	(1)	0	(1)	2	2	2	3
Three-axle	4	6	10	8	0	0	9	0	1	0	0	0	3
Average	10	9	2	4	0	(1)	(1)	0	(1)	1	1	1	2
Truck combinations:													
Tractor-truck and semitrailer	150	171	42	100	24	28	17	19	22	31	5	13	48
Truck and trailer	0	0	0	0	48	0	0	16	27	2	1	1	8
Average	150	170	42	100	25	28	16	19	22	26	3	9	45
Average, all trucks and truck combinations, 1947	40	50	12	29	10	9	5	5	7	5	2	3	14
Average, all trucks and truck combinations, 1946	21	42	10	23	6	5	3	1	4	5	2	3	10

[1] Less than 5 in 10,000.

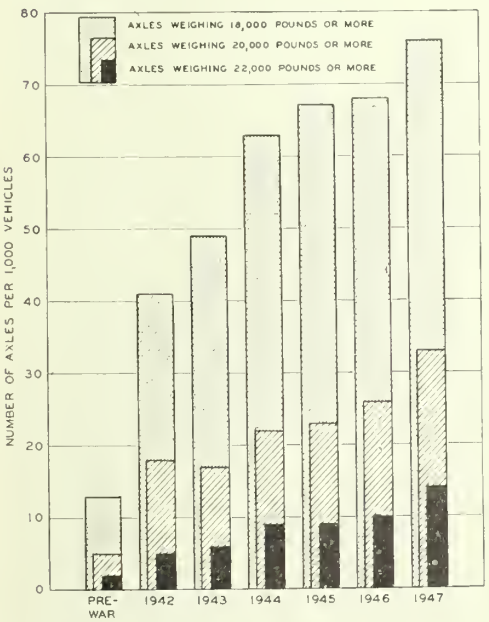


Figure 8.—Number of heavy axle loads per 1,000 trucks and truck combinations (empties included) in the summers of 1942–47 and a prewar year.

Table 12 shows that the high frequency of heavy axle loads in the Middle Atlantic and New England regions, as compared to the Pacific region, was probably not due to differences in enforcement practices. In the Pacific region, 58 of each 1,000 trucks and truck combinations were overloaded, according to State laws, as compared to 41 for the Middle Atlantic and 35 for the New England regions. The number of overloaded truck combinations per 1,000 vehicles of this type was 155 in the Pacific region, 123 in the Middle Atlantic region, and 129 in the New England region. The figures do show, however, that the overloading of truck combinations considerably in excess of legal limits was slightly less common in the Pacific region than in the Middle Atlantic and New England regions. The number of combinations overloaded more than 20 percent above the legal limit per 1,000 vehicles of that type amounted to 16 in the Pacific, 24 in the Middle Atlantic, and 19 in the New England region.

RECOMMENDED WEIGHT LIMITS

Uniform regulations concerning maximum allowable gross weights, axle weights, and axle-group weights have been adopted as a policy by the American Association of State Highway Officials and recommended to the various State governments for adoption.[6] This policy recommends that no axle shall carry a load in excess of 18,000 pounds and no group of axles shall carry a load in excess of amounts specified in a table of permissible weights based on the distance between the extremes of any group of axles.

The data indicate that the recommended axle-load limits are exceeded more frequently than are the other weight limits. In table 13 is shown the number of axles per 1,000 vehicles of various types that exceeded the axle-load limit of 18,000 pounds recommended by the A. A. S. H. O., and the number exceeding these limits by various percentages. This table emphasizes again the high frequency of heavy axle loads in the Middle Atlantic and New England regions.

(Text continued on page 157. Tables 12–15 are on pages 154–155.)

[6] *Policy concerning maximum dimensions, weights, and speeds of motor vehicles to be operated over the highways of the United States*, adopted April 1, 1946, by the American Association of State Highway Officials; published by the Association, 1946.

Table 12.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle, axle-group, or gross-weight laws in effect in the States by various percentages (maximum) of overload, in the summer of 1947*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 per-cent	10 per-cent	20 per-cent	30 per-cent	50 per-cent
New England:						
Panel and pick-up	----	---	---	---	---	--
Other 2-axle, 4-tired	7	---	---	---	---	--
Other 2-axle, 6-tired	19	16	13	6	5	(1)
Three-axle	69	52	43	4	---	--
Average, single-unit trucks	12	10	8	3	3	(1)
Traetor-truck and semitrailer	126	89	59	19	6	1
Truck and trailer	689	689	689	---	---	--
Average, truck combinations	129	92	62	19	6	1
Avg., all trucks and combinations	35	26	19	6	4	(1)
Middle Atlantic:						
Panel and pick-up	----	---	---	---	---	--
Other 2-axle, 4-tired	----	---	---	---	---	--
Other 2-axle, 6-tired	14	10	6	2	1	(1)
Three-axle	89	85	43	26	9	--
Average, single-unit trucks	11	9	5	2	1	(1)
Tractor-truck and semitrailer	124	87	54	24	10	3
Truck and trailer	----	---	---	---	---	--
Average, truck combinatious	123	86	54	24	10	3
Avg., all trucks and eombinations	41	29	18	8	3	1
South Atlantic:						
Panel and pick-up	----	---	---	---	---	--
Other 2-axle, 4-tired	---	---	---	---	---	--
Other 2-axle, 6-tired	4	3	3	2	(1)	--
Three-axle	34	34	30	25	15	5
Average, single-unit trucks	4	3	3	2	1	(1)
Tractor-truck and semitrailer	186	138	98	39	14	3
Truck and trailer	----	---	---	---	---	--
Average, truck combinations	185	138	98	39	14	3
Avg., all trucks and combiuations	53	39	28	12	4	1
East North Central:						
Panel and pick-up	----	---	---	---	---	--
Other 2-axle, 4-tired	---	---	---	---	---	--
Other 2-axle, 6-tired	1	1	1	---	---	--
Three-axle	277	204	158	136	68	--
Average, single-unit trueks	19	14	11	9	11	--
Tractor-truck and semitrailer	177	139	101	51	30	5
Truck and trailer	307	261	220	139	87	23
Average, truck combinations	184	146	108	56	33	6
Avg., all trueks and combinations	77	61	45	26	19	2
East South Central:						
Pauel and pick-up	1	---	---	---	---	--
Other 2-axle, 4-tired	----	---	---	---	---	--
Other 2-axle, 6-tired	4	3	1	1	---	--
Three-axle	88	71	60	48	39	36
Average, single-unit trueks	4	3	2	1	1	1
Traetor-truck and semitrailer	122	92	54	25	6	1
Truck and trailer	----	---	---	---	---	--
Average, truck combinations	122	92	54	25	6	1
Avg., all trucks and combinations	26	20	12	5	2	1
West North Central:						
Panel and pick-up	----	---	---	---	---	--
Other 2-axle, 4-tired	----	---	---	---	---	--
Other 2-axle, 6-tired	1	(1)	(1)	(1)	(1)	(1)
Three-axle	59	42	32	9	9	9
Average, single-unit trucks	1	1	(1)	(1)	(1)	(1)
Tractor-truck and semitrailer	183	129	79	30	13	1
Truck and trailer	----	---	---	---	---	--
Average, truek combinations	173	122	75	28	12	1
Avg., all trucks and combinations	38	27	16	6	3	(1)
West South Central:						
Panel and pick-up	----	---	---	---	---	--
Other 2-axle, 4-tired	----	---	---	---	---	--
Other 2-axle, 6-tired	3	2	2	1	---	--
Three-axle	29	18	18	---	---	--
Average, single-unit trucks	2	1	1	(1)	---	--
Tractor-truck and semitrailer	108	83	48	21	8	(1)
Truck and trailer	97	97	85	85	16	8
Average, truek combinations	107	84	50	25	8	(1)
Avg., all trucks and combinations	30	23	14	7	2	(1)
Mountain:						
Panel and piek-up	-- --	---	---	---	---	--
Other 2-axle, 4-tired	-- --	---	---	---	---	--
Other 2-axle, 6-tired	4	4	3	1	1	--
Three-axle	32	16	3	---	---	--
Average, single-unit trueks	3	2	1	(1)	(1)	--
Tractor-truck and semitrailer	148	105	86	32	11	4
Truck and trailer	134	93	62	16	3	--
Average, truck combinations	146	103	82	29	10	3
Avg., all trucks and combinations	28	20	15	5	2	1
Pacific:						
Panel and piek-up	----	---	---	---	---	--
Other 2-axle, 4-tired	----	---	---	---	---	--
Other 2-axle, 6-tired	10	8	6	4	1	--
Three-axle	26	12	12	2	---	--
Average, single-unit trueks	9	5	5	2	(1)	--
Tractor-truek and semitrailer	151	94	57	22	6	2
Truck and trailer	160	94	43	6	1	--
Average, truck combinations	155	94	51	16	4	1
Avg., all trucks and combinations	58	35	20	7	1	(1)
United States average:						
Panel and pick-up	(1)	---	---	---	---	--
Other 2-axle, 4-tired	(1)	---	---	---	---	--
Other 2-axle, 6-tired	5	4	3	1	(1)	(1)
Three-axle	78	57	44	31	16	2
Average, single-unit trucks	7	5	4	2	1	(1)
Traetor-truek and semitrailer	154	113	75	33	14	2
Truck and trailer	159	109	68	30	13	4
Average, truck combinations	154	113	74	33	14	2
Avg., all trueks and combinations	46	34	23	10	4	1

[1] Less than 5 per 10,000.

Table 13.—*Number of axles, per 1,000 loaded and empty trucks and truck combinations, that exceeded the permissible axle-load limit of 18,000 pounds recommended by the A. A. S. H. O. by various percentages of overload, in the summer of 1947*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 per-cent	10 per-cent	20 per-cent	30 per-cent	50 per-eent
New England:						
Panel and pick-up	----	---	---	---	---	---
Other 2-axle, 4-tired	10	---	---	---	---	---
Other 2-axle, 6-tired	39	38	32	20	12	6
Three-axle	91	56	49	8	---	---
Average, single-unit trucks	23	21	18	10	6	3
Tractor-truck and semitrailer	451	371	285	161	72	8
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	449	369	284	160	72	8
Avg., all trucks and combinations	108	90	71	40	19	4
Middle Atlantic:						
Panel and pick-up	----	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	32	29	24	15	6	1
Three-axle	46	34	31	6	---	---
Average, single-unit trucks	19	17	14	8	3	1
Tractor-truck and semitrailer	503	418	327	195	105	25
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	500	415	325	194	104	25
Avg., all trucks and combinations	146	122	96	57	30	7
South Atlantic:						
Panel and pick-up	----	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	5	4	3	2	1	---
Three-axle	15	10	10	10	5	5
Average, single-unit trucks	3	3	2	2	1	(1)
Traetor-truck and semitrailer	259	185	126	45	17	5
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	258	184	126	45	17	5
Avg., all trucks and combinations	71	52	35	14	5	1
East North Central:						
Panel and piek-up	----	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	1	1	1	---	---	---
Three-axle	119	23	23	23	---	---
Average, single-unit trucks	8	2	2	2	---	---
Tractor-truck and semitrailer	199	136	80	26	9	(1)
Truck and trailer	520	347	225	76	18	---
Average, truck combinations	217	148	88	29	10	(1)
Avg., all trucks and combinations	82	54	32	12	4	(1)
East South Central:						
Panel and pick-up	1	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	4	3	1	1	---	---
Three-axle	18	18	18	9	---	---
Average, single-unit trucks	3	2	1	1	---	---
Tractor-truck and semitrailer	163	114	69	26	5	1
Truck and trailer	---	---	---	---	---	---
Average, truek combinations	163	114	69	26	5	1
Avg., all trucks and combinations	33	23	14	6	1	(1)
West North Central:						
Panel and pick-up	----	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	1	1	(1)	(1)	(1)	---
Three-axle	23	11	9	9	---	---
Average, single-unit trucks	1	1	(1)	(1)	(1)	---
Tractor-truck and semitrailer	207	158	77	23	6	1
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	196	149	73	22	6	1
Avg., all trucks and combinations	43	33	16	5	1	(1)
West South Central:						
Panel and pick-up	----	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	3	2	2	1	---	---
Three-axle	35	35	18	---	---	---
Average, single-unit trueks	2	1	1	(1)	---	---
Tractor-truck and semitrailer	129	98	57	22	7	(1)
Truck and trailer	145	145	56	16	16	8
Average, truck combinations	130	101	57	22	8	(1)
Avg., all trucks and combinations	37	28	16	6	2	(1)
Mountain:						
Panel and pick-up	----	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	4	4	3	1	1	---
Three-axle	32	7	2	---	---	---
Average, single-unit trucks	3	2	1	(1)	(1)	---
Traetor-truck and semitrailer	154	108	72	31	13	3
Truck and trailer	80	29	17	3	---	---
Average, truck combinations	142	96	63	27	11	3
Avg., all trucks aud combinations	28	19	12	5	2	1
Pacific:						
Panel and piek-up	----	---	---	---	---	---
Other 2-axle, 4-tired	---	---	---	---	---	---
Other 2-axle, 6-tired	10	8	6	4	1	---
Three-axle	14	5	1	---	---	---
Average, single-unit trucks	6	4	2	1	(1)	---
Tractor-truck and semitrailer	125	65	37	14	4	1
Truck and trailer	98	39	16	2	---	---
Average, truck combinations	114	55	29	9	2	1
Avg., all trucks and combinations	42	21	11	4	1	(1)
United States average:						
Panel and pick-up	(1)	---	---	---	---	---
Other 2-axle, 4-tired	(1)	---	---	---	---	---
Other 2-axle, 6-tired	9	8	6	4	2	(1)
Three-axle	37	14	11	7	1	1
Average, single-unit trucks	6	5	4	2	1	(1)
Tractor-truck and semitrailer	239	178	119	54	24	5
Truck and trailer	146	84	44	12	4	1
Average, truck combinations	231	169	112	50	22	5
Avg., all trucks and combinations	66	49	33	15	7	1

[1] Less than 5 per 10,000.

Table 14.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle-group loads recommended by the A. A. S. H. O. by various percentages of overload, in the summer of 1947*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 per-cent	10 per-cent	20 per-cent	30 per-cent	50 per-cent
New England:						
Panel and pick-up	-----	---	---	-----	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	3	(1)	(1)	---	---	--
Three-axle	117	100	44	8	4	--
Average, single-unit trucks	5	3	1	(1)	(1)	--
Tractor-truck and semitrailer	85	52	36	11	5	(1)
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	85	52	36	11	5	(1)
Avg., all trucks and combinations	21	13	8	2	1	(1)
Middle Atlantic:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	1	(1)	(1)	---	---	--
Three-axle	40	31	24	6	---	--
Average, single-unit trucks	2	1	1	(1)	---	--
Tractor-truck and semitrailer	117	83	63	27	15	5
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	116	83	63	27	15	5
Avg., all trucks and combinations	32	23	17	7	4	1
South Atlantic:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	-----	---	---	---	---	--
Three-axle	19	15	5	---	---	--
Average, single-unit trucks	1	1	(1)	---	---	--
Tractor-truck and semitrailer	37	25	19	9	3	1
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	37	25	19	9	3	1
Avg., all trucks and combinations	11	7	5	2	1	(1)
East North Central:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	-----	---	---	---	---	--
Three-axle	-----	---	---	---	---	--
Average, single-unit trucks	-----	---	---	---	---	--
Tractor-truck and semitrailer	99	78	60	27	9	1
Truck and trailer	364	358	283	237	179	63
Average, truck combinations	114	94	73	39	19	5
Avg., all trucks and combinations	40	33	26	14	7	2
East South Central:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	-----	---	---	---	---	--
Three-axle	7	7	---	---	---	--
Average, single-unit trucks	(1)	(1)	---	---	---	--
Tractor-truck and semitrailer	13	9	3	(1)	(1)	--
Truck and trailer	-----	---	---	---	---	--
Average, truck combinations	13	9	3	(1)	(1)	--
Avg., all trucks and combinations	2	2	1	(1)	(1)	--
West North Central:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	(1)	(1)	(1)	(1)	(1)	--
Three-axle	13	11	9	9	9	--
Average, single-unit trucks	(1)	(1)	(1)	(1)	(1)	--
Tractor-truck and semitrailer	74	50	34	16	5	(1)
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	70	47	32	15	5	(1)
Avg., all trucks and combinations	15	10	7	3	1	(1)
West South Central:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	-----	---	---	---	---	--
Three-axle	12	---	---	---	---	--
Average, single-unit trucks	(1)	---	---	---	---	--
Tractor-truck and semitrailer	19	13	8	4	1	(1)
Truck and trailer	32	32	32	32	32	16
Average, truck combinations	20	14	9	6	3	1
Avg., all trucks and combinations	5	4	2	2	1	(1)
Mountain:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	-----	---	---	---	---	--
Three-axle	10	9	5	---	---	--
Average, single-unit trucks	(1)	(1)	(1)	---	---	--
Tractor-truck and semitrailer	78	65	46	22	10	1
Truck and trailer	125	99	68	20	---	--
Average, truck combinations	85	70	49	22	8	1
Avg., all trucks and combinations	15	12	9	4	1	(1)
Pacific:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	-----	---	---	---	---	--
Three-axle	14	13	5	---	---	--
Average, single-unit trucks	3	3	1	---	---	--
Tractor-truck and semitrailer	187	143	103	33	13	3
Truck and trailer	242	173	77	12	1	--
Average, truck combinations	209	155	93	25	8	2
Avg., all trucks and combinations	72	54	32	8	3	1
United States average:						
Panel and pick-up	-----	---	---	---	---	--
Other 2-axle, 4-tired	-----	---	---	---	---	--
Other 2-axle, 6-tired	(1)	(1)	(1)	(1)	(1)	--
Three-axle	17	14	7	1	(1)	--
Average, single-unit trucks	1	1	(1)	(1)	(1)	--
Tractor-truck and semitrailer	77	57	42	17	7	1
Truck and trailer	209	163	91	42	26	9
Average, truck combinations	89	67	46	19	9	2
Avg., all trucks and combinations	25	19	12	5	2	1

[1] Less than 5 per 10,000.

Table 15.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible motor-vehicle loads recommended by the A. A. S. H. O. by various percentages (maximum) of overload, in the summer of 1947*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 per-cent	10 per-cent	20 per-cent	30 per-cent	50 per-cent
New England:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	10	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	39	38	32	20	12	6
Three-axle	137	105	62	8	4	-----
Average, single-unit trucks	24	22	18	10	6	3
Tractor-truck and semitrailer	300	252	198	120	56	7
Truck and trailer	-----	-----	-----	-----	-----	-----
Average, truck combinations	299	251	197	119	56	7
Avg., all trucks and combinations	79	68	54	32	16	4
Middle Atlantic:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	32	29	24	15	6	1
Three-axle	56	53	44	6	-----	-----
Average, single-unit trucks	19	18	15	8	3	1
Tractor-truck and semitrailer	328	277	225	142	80	22
Truck and trailer	-----	-----	-----	-----	-----	-----
Average, truck combinations	326	275	224	141	80	22
Avg., all trucks and combinations	100	86	70	43	23	7
South Atlantic:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	5	4	3	2	1	-----
Three-axle	24	20	10	10	5	5
Average, single-unit trucks	4	3	2	2	1	(1)
Tractor-truck and semitrailer	199	147	108	45	18	5
Truck and trailer	-----	-----	-----	-----	-----	-----
Average, truck combinations	198	147	108	45	18	5
Avg., all trucks and combinations	56	42	30	14	6	1
East North Central:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	1	1	1	-----	-----	-----
Three-axle	142	23	23	23	-----	-----
Average, single-unit trucks	10	2	2	2	-----	-----
Tractor-truck and semitrailer	184	137	102	41	15	1
Truck and trailer	369	358	283	237	179	52
Average, truck combinations	195	150	112	52	24	4
Avg., all trucks and combinations	75	54	41	20	8	1
East South Central:						
Panel and pick-up	1	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	4	3	1	1	-----	-----
Three-axle	34	31	18	9	-----	-----
Average, single-unit trucks	3	2	1	1	-----	-----
Tractor-truck and semitrailer	122	91	56	22	4	1
Truck and trailer	-----	-----	-----	-----	-----	-----
Average, truck combinations	122	91	56	22	4	1
Avg., all trucks and combinations	25	19	11	5	1	(1)
West North Central:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	1	(1)	(1)	(1)	(1)	-----
Three-axle	32	27	25	9	9	-----
Average, single-unit trucks	1	(1)	(1)	(1)	(1)	-----
Tractor-truck and semitrailer	175	125	76	26	8	1
Truck and trailer	-----	-----	-----	-----	-----	-----
Average, truck combinations	165	118	72	25	8	1
Avg., all trucks and combinations	36	25	16	5	2	(1)
West South Central:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	3	2	2	1	-----	-----
Three-axle	29	18	18	18	-----	-----
Average, single-unit trucks	2	1	1	1	-----	-----
Tractor-truck and semitrailer	100	79	45	21	7	1
Truck and trailer	97	97	32	32	32	16
Average, truck combinations	100	80	44	22	9	2
Avg., all trucks and combinations	28	22	13	7	2	1
Mountain:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	4	4	3	1	1	-----
Three-axle	30	14	5	-----	-----	-----
Average, single-unit trucks	2	2	1	(1)	(1)	-----
Tractor-truck and semitrailer	157	119	86	41	17	4
Truck and trailer	161	136	98	28	-----	-----
Average, truck combinations	158	122	88	39	14	3
Avg., all trucks and combinations	30	23	16	7	2	1
Pacific:						
Panel and pick-up	-----	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	-----	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	10	8	6	4	1	-----
Three-axle	17	13	6	-----	-----	-----
Average, single-unit trucks	7	5	3	1	(1)	-----
Tractor-truck and semitrailer	217	164	117	39	14	3
Truck and trailer	300	196	90	14	1	-----
Average, truck combinations	250	177	106	29	9	2
Avg., all trucks and combinations	88	63	38	10	3	1
United States average:						
Panel and pick-up	(1)	-----	-----	-----	-----	-----
Other 2-axle, 4-tired	(1)	-----	-----	-----	-----	-----
Other 2-axle, 6-tired	9	7	6	4	2	(1)
Three-axle	46	23	15	7	1	1
Average, single-unit trucks	7	5	4	2	1	(1)
Tractor-truck and semitrailer	195	151	110	52	23	5
Truck and trailer	255	187	102	43	26	8
Average, truck combinations	200	154	109	51	23	5
Avg., all trucks and combinations	59	45	32	15	7	1

[1] Less than 5 per 10,000.

Trends in Motor-Vehicle Travel, 1947

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by G. P. St. CLAIR, Chief of Division

Total travel of motor vehicles in 1947 is estimated as 371 billion vehicle-miles, of which 187 billion was on rural highways and 184 billion on urban roads and streets. Passenger cars accounted for more than 300 billion vehicle-miles of the total, busses 4 billion, and trucks and combinations 66 billion. In every category these totals were greater than in any previous year. Total travel in 1947 exceeded that of 1946 by 9 percent and was 11 percent more than in 1941. The increase in travel of trucks and combinations was more than twice that of passenger cars, the former rising in 1947 to 18 percent above 1946, the latter 7 percent.

The average vehicle traveled 9,831 miles in 1947, using 746 gallons of gasoline at a rate of 13.19 miles per gallon. After the wartime decline, average annual automobile travel rose in 1946 to 7 percent more than the 1941 level, but in 1947 dropped back to 5 percent above 1941. Average annual truck travel, while showing a steady recovery, had in 1947 only reached 93 percent of the 1941 level.

In addition to the 1947 data, this article reports estimated total motor-vehicle travel in the United States for the years 1921–47, with a segregation between rural and urban traffic from 1929 on. In these 26 years, total annual travel has risen from 55 billion to 371 billion vehicle-miles, an increase of 574 percent. Rural travel has gradually risen in proportion from 46 to 50 percent of the total.

Although a respectable body of current data underlies the annual travel estimates, it is still necessary to base them fundamentally on the patterns determined by the comprehensive traffic surveys made circa 1936. As the base period becomes more remote the hazards of this procedure increase. A new set of comprehensive surveys is needed to provide a better foundation for the many problems in highway planning, construction, and operation that lie ahead.

THE SERIES of classified estimates of motor-vehicle travel in the United States reported in two previous articles in PUBLIC ROADS [1] is carried forward in this article for the year 1947. The procedures used in making these estimates were the same as those described in the two previous reports, the principal factors controlling the calculations being (1) the annual estimates of rural-road traffic described in another article in this issue of PUBLIC ROADS; (2) the annual reports of highway use of motor fuel; and (3) reported motor-vehicle registrations. As in the previous calculations, the 1947 estimates were based on indicated changes from a previous year, the year 1946 being used as the base year in this instance.

Table 1 reports, for the various classes of motor vehicles, the estimates for 1947 of rural, urban, and total vehicle-miles traveled, average miles traveled and motor fuel consumed per vehicle, and average travel per gallon of motor fuel consumed. The numbers of registered vehicles, as modified for the purpose of these estimates, are also given. The total travel of motor vehicles in 1947 is estimated as 371 billion vehicle-miles, of which 187 billion were traveled on rural highways and 184 billion on urban highways and streets. Passenger cars traveled approximately 300 billion vehicle-miles, busses over 4 billion, and trucks and combinations 66 billion. For all vehicle types and for both rural and urban travel, these totals were greater than in any previous year.

Table 2 gives a comparison of the 1947 vehicle-mile estimates with those for 1941 and 1946. Total travel in 1947 was 11.2 percent greater than in 1941, the maximum prewar year, and was 8.8 percent greater than in 1946.

The relative increase of rural travel in 1947 over that of 1946 was slightly greater than that of urban travel. A similar but much greater contrast observed in the comparison of the 1946 with the 1945 estimates [1] was attributed to the recovery from the restrictions of the war years, during which rural travel suffered much greater reductions than did urban travel. The two successive comparisons (expressed in percent) are as follows:

	1946 over 1945	*1947 over 1946*
Increase of rural travel	43. 2	9. 3
Increase of urban travel	31. 1	8. 3

In the comparison with 1941 values table 2 shows that the relative increase in urban travel, 12.5 percent, was greater than that in rural travel, which was 9.9 percent. A later section of this report deals with the trends, over a period of years, in the relation between rural and urban travel.

Both comparisons given in table 2 indicate that the relative increase in the travel of trucks

Table 1.—*Classified estimate of motor-vehicle travel in the United States in the calendar year 1947*

Vehicle type	Motor-vehicle travel			Number of registered vehicles [1]	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Rural travel	Urban travel	Total travel			Total [2]	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles*
Passenger vehicles:								
Passenger cars [3]	143, 662	156, 620	300, 282	30, 872	9, 727	20, 086	651	14 95
Busses:								
Commercial	1, 480	2, 080	3, 560	89	40, 000	712	8, 000	5. 00
School and nonrevenue	621	70	691	87	7, 989	66	770	10. 37
All busses	2, 101	2, 150	4, 251	176	24, 221	778	4, 436	5. 46
All passenger vehicles	145, 763	158, 770	304, 532	31, 048	9, 809	20, 864	672	14. 60
Trucks and combinations	40, 771	25, 318	66, 089	6, 650	9, 939	7, 243	1, 089	9. 13
All motor vehicles	186, 534	184, 088	370, 622	37, 698	9, 831	28, 107	746	13. 19

[1] These registration totals differ from those given in Public Roads Administration table MV–1 for 1947 because of the following adjustments: (1) Approximate correction for defective classification in 4 States, as described in footnotes 9, 10, 13, and 14 of that table; (2) inclusion of publicly owned vehicles, listed separately in table MV–1; (3) reduction of private and commercial truck registrations by 2.5 percent to allow for registrations in more than 1 State; and (4) substitution of bus totals as estimated by the bus industry to afford a complete segregation of commercial busses from school and nonrevenue busses and to allow for duplication because of registrations in more than 1 State.

[2] Total highway use of motor fuel in 1947 is given as 28,216 million gallons in Public Roads Administration table G–21. For this analysis there was deducted from that total 109 million gallons estimated use by motorcycles (250 gallons per motorcycle).

[3] Including taxicabs.

[1] *Trends in motor-vehicle travel, 1936 to 1945*, PUBLIC ROADS, vol. 24, No. 10, Oct.-Nov.-Dec. 1946, and *Trends in motor-vehicle travel, 1946*, PUBLIC ROADS, vol. 25, No. 3, March 1948.

Table 2.—*Increase in motor-vehicle travel, 1947 compared with 1941 and 1946*

Vehicle type	Percentage increase of travel—					
	1947 over 1941			1947 over 1946		
	Rural	Urban	Total	Rural	Urban	Total
Passenger vehicles [1]	7.7	10.9	9.3	7.2	6.9	7.0
Trucks and combinations	18.5	23.6	20.4	17.9	17.5	17.7
All motor vehicles	9.9	12.5	11.2	9.3	8.3	8.8

[1] Includes both passenger cars and busses.

and combinations has been more than twice that of passenger vehicles. The estimated travel of trucks and combinations in 1947 was 20.4 percent more than their travel in 1941, and 17.7 percent in excess of their 1946 travel; the corresponding figures for passenger vehicles being 9.3 and 7.0 percent, respectively. This remarkable increase in truck traffic undoubtedly reflects the sustained and increasing volume of business activity in the postwar years. Reference to the article *Traffic Trends on Rural Roads in 1947*, also published in this issue, will reveal the fact that the ton-miles of freight operation on main roads has increased even more rapidly than the vehicle-miles of truck travel.[2]

AVERAGE ANNUAL MILEAGES

Further comparisons of the travel characteristics of passenger cars and trucks are given in figure 1 and table 3. Figure 1 compares the average miles per year traveled by passenger cars, as given by these annual estimates, with the corresponding values for trucks and combinations. It will be observed that, for the period from 1936 to 1941, the average annual mileages of trucks and combinations were well above those of passenger cars, and that for both vehicle types the values were increasing at rather steady rates from year to year. As a result of gasoline rationing and other restrictions during the war period, the average annual mileages of both vehicle types fell off sharply.

[2] See figures 4, 5, and 6 of above-cited article, and text, pp. 148–152.

Because of the necessities of wartime freight transportation, the restrictions on passenger cars were much more severe than those on trucks; and the average mileage of passenger cars fell off to about 6,200 miles per year in 1943, whereas the average for trucks fell only slightly below 9,000 miles in the minimum year, 1944.

The trends during the postwar years appear to have been very different. The average mileage of passenger cars recovered very sharply in 1945 and 1946; the value for the latter year being nearly 10,000 miles per vehicle, a figure considerably in excess of the prewar averages. In 1947 the value fell to 9,727 miles per vehicle. In the case of trucks and combinations, there has been a steady recovery, but the 1947 value, 9,939 miles per vehicle, is considerably below that of 10,739 miles in 1941 and slightly below that for 1936.

Some light on the peculiarities displayed in figure 1 may be obtained by an examination of the data set forth in table 3, which gives for passenger cars and for trucks and combinations the total vehicle-miles traveled, numbers registered, and average miles per vehicle for the years 1936–47. Each of these quantities is indexed against its value for the year 1941.

Turning first to passenger cars, we find that the total travel of these vehicles in 1947 was 8.9 percent above the 1941 value. The number registered, however, was only 4 percent above 1941. The tremendous unsatisfied demand for passenger automobiles is a matter of common knowledge. Among the causes for this great demand, two principal factors can be discerned. First, the abnormal age distribution of passenger cars resulting from the wartime cessation of production has produced a very large need for replacements. Second, the great rise in national income, and the penetration of relatively high incomes among what were formerly low-income groups in the labor and farm population has greatly broadened the base for the purchase of new and used passenger cars. Automobile production was slow in gaining headway during the first two postwar years, and only in 1948 has anything approaching a maximum production rate been reached. In the face of great demand and short supply it is natural that the passenger cars on hand should be overworked, so to speak, with a resulting increase of average passenger-car mileage to values well above those of the prewar years.

Evidence that this effect was somewhat diminished in 1947 is found in the fact that the estimated average annual mileage of these vehicles dropped from 9,942 miles in 1946 to 9,727 miles in 1947. This drop may be partially explained by the fact that a greater number of vehicles were driven less than 12 months in 1947 than were so driven in 1946. This fact becomes obvious when it is pointed out that there were more than 3 million new passenger-car registrations in 1947, whereas there were approximately 1.8 million in 1946.[3] It is probable, however, that with a materially increased supply of passenger cars in 1947, the tendency to drive the average car a greater than normal mileage had begun to diminish. It is not unlikely that the trend toward somewhat decreased average annual mileage of passenger cars will continue for a year or two as a result of the greatly increased production of these vehicles.

The data for trucks indicate a very different situation. It is shown in table 3 that the travel of trucks and combinations in 1947 was 20.4 percent above the 1941 value. The

[3] New passenger-car registrations, as reported by R. L. Polk and Company, and quoted in the Automobile Manufacturers Association publication *Automobile Facts and Figures*, 1948 edition, p. 13, were 1,815,196 in 1946 and 3,167,231 in 1947.

Table 3.—*Comparison of total vehicle-miles, registrations, and average miles per vehicle, for passenger cars and for trucks and combinations, 1936–47*

Year	Passenger cars						Trucks and combinations					
	Estimate of total vehicle-miles traveled [1]		Number registered [1]		Average miles per vehicle [1]		Estimate of total vehicle-miles traveled [1]		Number registered [1]		Average miles per vehicle [1]	
	Amount	Index on 1941 value	Amount	Index on 1941 value	Amount	Index on 1941 value	Amount	Index on 1941 value	Amount	Index on 1941 value	Amount	Index on 1941 value
	Millions		*Thousands*				*Millions*		*Thousands*			
1936	208,654	75.7	24,201	81.5	8,622	92.9	41,107	74.9	4,071	79.6	10,098	94.0
1937	223,467	81.1	25,490	85.9	8,767	94.4	44,151	80.4	4,301	84.1	10,264	95.6
1938	224,174	81.3	25,272	85.1	8,871	95.5	44,495	81.1	4,285	83.8	10,383	96.7
1939	235,629	85.5	26,252	88.4	8,976	96.7	47,219	86.0	4,495	87.9	10,504	97.8
1940	249,559	90.5	27,488	92.6	9,079	97.8	49,927	91.0	4,699	91.9	10,624	98.9
1941	275,685	100.0	29,691	100.0	9,285	100.0	54,891	100.0	5,112	100.0	10,739	100.0
1942	218,239	79.2	27,970	94.2	7,803	84.0	45,727	83.3	4,762	93.2	9,602	89.4
1943	161,788	58.7	26,005	87.6	6,221	67.0	41,594	75.8	4,611	90.2	9,021	84.0
1944	166,073	60.2	25,562	86.1	6,497	70.0	41,708	76.0	4,642	90.8	8,984	83.7
1945	199,185	72.3	25,789	86.9	7,724	83.2	45,909	83.6	4,956	96.9	9,264	86.3
1946	280,457	101.7	28,209	95.0	9,942	107.1	56,146	102.3	5,839	114.2	9,615	89.5
1947	300,282	108.9	30,872	104.0	9,727	104.8	66,089	120.4	6,650	130.1	9,939	92.6

[1] Values as given in *Trends in motor-vehicle travel, 1936 to 1945*, Public Roads, Oct.-Nov.-Dec. 1946; *Trends in motor-vehicle travel, 1946*, Public Roads, March 1948; and in table 1 of this article.

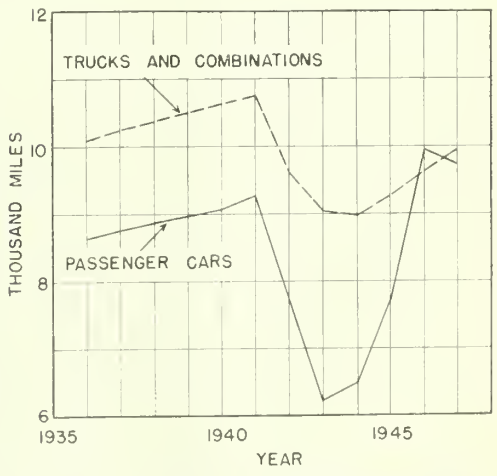


Figure 1.—Trends in the average miles per vehicle traveled by passenger cars and by trucks and combinations.

number registered, on the other hand, was 30.1 percent above 1941. In contrast to the case of passenger cars, we find the supply of trucks, as evidenced by registrations, showing a greater relative increase than the demand for truck transportation as evidenced by vehicle-miles traveled. Although the truck-production figures indicate a continuing healthy demand for freight vehicles, it is quite evident that the shortage of supply is by no means as acute as in the case of passenger cars. It should be borne in mind that the increase in truck travel over 1941 values was more than twice the increase in passenger-car travel, and that, in terms of ton-miles of operation, the increase was even greater. In spite of these great increases, the rise in truck registrations was such that the large volume of increased business could be carried at an average mileage per vehicle lower than that of the prewar years. It is probable also that a considerable part of the increased truck registrations include those of farmers, small business men, and trucking operators new to the field, all of whom would have a tendency toward low-mileage operation.

Although the explanations given above tend to confirm the comparisons given in figure 1 and table 3, there may be some disposition to question whether the accuracy of the estimates is such as to make these findings entirely acceptable. It must be acknowledged that insufficient data are available to produce estimates which could be accepted as highly accurate. It is necessary to use certain procedures and certain assumptions that are not fully supported, and to carry forward trends from year to year which may involve the continuation of past inaccuracies. It is found, however, that where options exist in making the estimates, they do not lead to large deviations in the final results. If an arbitrary adjustment is attempted in order to eliminate an apparent inconsistency in one item, compensating adjustments in other items will produce greater inconsistencies than the one it is desired to remove. Although uncertainties as to the accuracy of the estimates must continue to exist until more adequate data are available, it is believed the findings are reasonably consistent and in harmony with discernible trends.

VEHICLE-MILE ESTIMATES, 1921–47

In table 4 there are given values of the estimated total motor-vehicle travel in the United States for the years 1921–47. A segregation of these values between rural and urban traffic is given for the years 1929–47. Values from 1936 through 1946 were given in the two PUBLIC ROADS articles previously referred to.[4] The values for the earlier years have been given some currency but have not previously been published.

Because of the fact that comprehensive studies of traffic and motor-vehicle use were not made prior to the State-wide highway planning survey period, for which 1936 is generally taken as the base year, it is not possible at this late date to make accurate estimates of motor-vehicle travel for years prior to 1936. Estimates of total travel given in table 4 were obtained by applying to the values of total highway use of motor fuel given in the annual motor-fuel usage analysis (Public Roads Administration table G–21) the value of 13.984 miles per gallon, derived from the vehicle-mile estimates for the year 1936. Although this procedure is open to question, there exists no volume of data whereby an accurate trend in the variation of average miles per gallon for those earlier years could be developed. The values given in table 4 are without doubt sufficiently accurate for the purposes for which estimates of vehicle-miles traveled in years prior to 1936 are needed.

The values of rural and urban travel given in table 4 were derived by extending backward to 1929 the trend in percentage relation of rural travel to total travel indicated by the analysis which led to the estimates of vehicle-miles for the years 1936–41. Such a procedure is also of questionable accuracy; and it was felt that this estimated segregation between rural and urban travel should not be carried to years earlier than 1929. No classification of these earlier estimates by type of vehicle has been made. Reasonable values for a few years prior to 1936 could probably be estimated, but it is felt that no very useful purpose would be served by making such estimates. The trend curves for total travel from 1921 through 1947 and for rural and urban travel from 1929 through 1947 are given in figure 2.

[4] See footnote 1, p. 156.

COMPARISON OF TRENDS IN MOTOR-FUEL USE AND RURAL HIGHWAY TRAVEL

Table 4 gives, for each year from 1929 through 1947, the percentage distribution between rural and urban travel. These values indicate a steadily increasing proportion of rural travel to total travel from 1929 through 1941. The question may be raised whether this indication reflects the findings of actual data or whether assumptions have been made in annual estimates which have produced an artificial trend not confirmed by facts. The comparisons given in table 5 and illustrated in figure 3 throw some light on this question.

Table 5 gives, for the years 1929–47, the Nation-wide totals of highway use of motor fuel. These values are also expressed in the form of an index and compared with the cor-

Table 4.—*Estimate of motor-vehicle travel in the United States, 1921–47* [1]

Year	Rural		Urban		Total	
	Amount	Percentage	Amount	Percentage	Amount	Percentage
	Million vehicle-miles		*Million vehicle-miles*		*Million vehicle-miles*	
1921	---	---	---	---	55,027	---
1922	---	---	---	---	67,697	---
1923	---	---	---	---	84,995	---
1924	---	---	---	---	104,838	---
1925	---	---	---	---	122,346	---
1926	---	---	---	---	140,735	---
1927	---	---	---	---	158,453	---
1928	---	---	---	---	172,856	---
1929	90,311	45.7	107,409	54.3	197,720	100.0
1930	95,118	46.1	111,202	53.9	206,320	100.0
1931	100,571	46.5	115,580	53.5	216,151	100.0
1932	94,151	47.0	106,366	53.0	200,517	100.0
1933	95,064	47.4	105,578	52.6	200,642	100.0
1934	103,050	47.8	112,513	52.2	215,563	100.0
1935	110,241	48.2	118,327	51.8	228,568	100.0
1936	122,678	48.7	129,450	51.3	252,128	100.0
1937	132,038	48.9	138,072	51.1	270,110	100.0
1938	134,913	49.8	136,264	50.2	271,177	100.0
1939	143,149	50.2	142,253	49.8	285,402	100.0
1940	152,150	50.4	149,993	49.6	302,143	100.0
1941	169,805	50.9	163,591	49.1	333,396	100.0
1942	128,861	48.2	138,235	51.8	267,096	100.0
1943	97,757	47.3	108,990	52.7	206,747	100.0
1944	100,830	47.7	110,750	52.3	211,580	100.0
1945	119,183	47.9	129,743	52.1	248,926	100.0
1946	170,606	50.1	170,049	49.9	340,655	100.0
1947	186,534	50.3	184,088	49.7	370,622	100.0

[1] For years prior to 1936 values of total motor-vehicle travel were extrapolated with the aid of motor-fuel consumption data, by assuming a constant value for average miles per gallon, e. g., 13.984 miles per gallon, the value derived in making the estimate for 1936.

Table 5.—*Comparison of trends in highway use of motor fuel and rural highway travel, 1929–47*

Year	Highway use of motor fuel [1]	Indexes relative to 1936 values	
		Highway use of motor fuel	Rural highway travel [2]
	Million gallons		
1929	14,139	78.1	73.6
1930	14,754	81.5	77.5
1931	15,457	85.4	82.0
1932	14,339	79.2	76.7
1933	14,348	79.3	77.5
1934	15,415	85.2	84.0
1935	16,345	90.3	89.9
1936	18,099	100.0	100.0
1937	19,455	107.5	107.6
1938	19,612	108.4	110.0
1939	20,714	114.4	116.7
1940	22,001	121.6	124.0
1941	24,192	133.7	138.4
1942	19,940	110.2	105.0
1943	16,004	88.4	79.7
1944	16,430	90.8	82.2
1945	19,149	105.8	97.2
1946	25,649	141.7	139.1
1947	28,216	155.9	152.1

[1] See *Highway Statistics, Summary to 1945*, Public Roads Administration, 1947, p. 5; and table G–21, 1947.
[2] Derived from table 4, second column.

responding index of the volume of rural travel, calculated from the data given in table 4. The index values are based on the year 1936, which is the year with which the annual vehicle-mile estimates began, and is generally taken as the base year of the comprehensive planning-survey studies. Both sets of indexes are plotted in the upper panel of figure 3. It will be observed that, for the years 1937–41, there is a gradual upward divergence of the rural travel index from that of total highway use of motor fuel. In other words, the slope of the rural traffic line is appreciably steeper than the slope of the motor-fuel consumption line.

These two sets of index values were derived from entirely independent data. The rural travel estimates are taken without change from the annual calculations of rural travel based chiefly on the fixed automatic recorder data. The values for total highway use of motor fuel are obtained by an analysis of the reports of motor-fuel consumption submitted by the several States. The motor-fuel data can be taken as representative of total motor-vehicle travel, both rural and urban. This being the case, the increase of the rural-travel index relative to the motor-fuel-use index during the period 1931–41 is a definite indication that the percentage relation of rural travel to total travel was rising during these years.

The objection might be raised that changes in average miles per gallon, as between rural and urban use of motor vehicles, could account for the difference in the two trend lines. The loadometer studies on main rural roads indicate that the gross loads and carried loads of trucks and combinations increased materially during the period under study; but this factor, rather than accounting for the difference in trends, would work in the opposite direction, since it would imply an increase in motor-fuel consumption on rural roads, relative to the total volume of rural travel.

This positive indication for the period 1936–41 lends support to the extrapolated values for rural travel prior to 1936, which carry backward the same trend. Index values for the two graphs in figure 3 cross at the year 1936, but the steeper slope of the index of rural highway travel is plainly evident. The percentage relations of rural travel to total travel are shown graphically in the lower panel of figure 3 and they indicate a gradual but steady rise in the relative amount of rural travel from 1929 through 1941.

The indicated trend in the relation of rural to urban travel is consistent with the generally known facts of highway and motor-vehicle history. In the early days of the motor vehicle its ownership and use was confined largely to the cities and their immediate surroundings. As the network of improved rural roads gradually developed, rural ownership increased and the radius of operation of urban vehicles was greatly enlarged, with the result that the volume of rural travel began gradually to overtake that of urban travel. The planning-survey road-use studies, conducted in a large number of States during the period 1935–39, indicated a Nation-wide distribution of somewhat more than 50 percent rural travel.

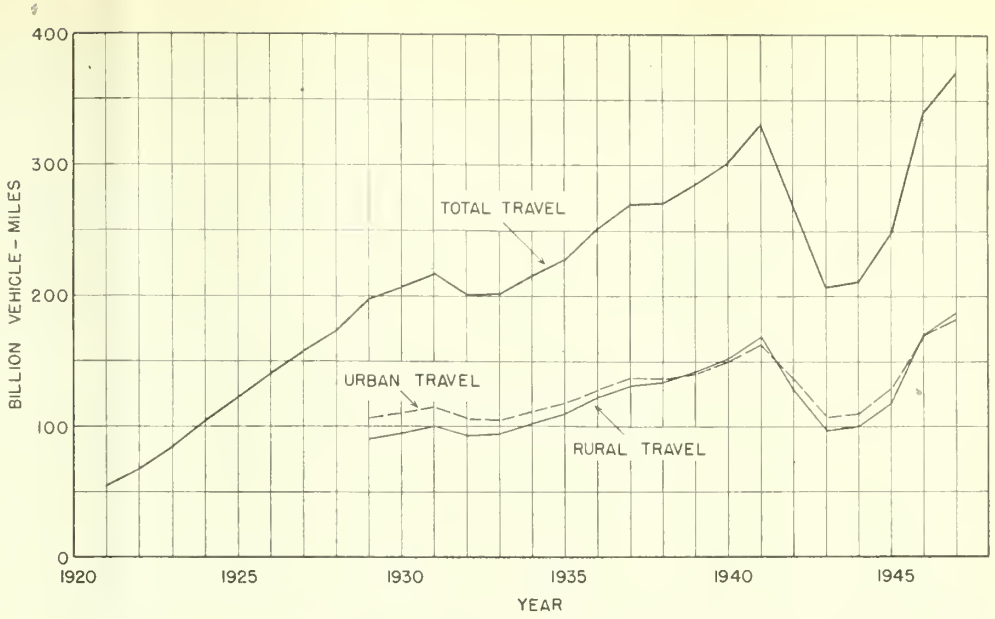


Figure 2.—Estimated total motor-vehicle travel, 1921–47; estimated rural and urban travel, 1929–47.

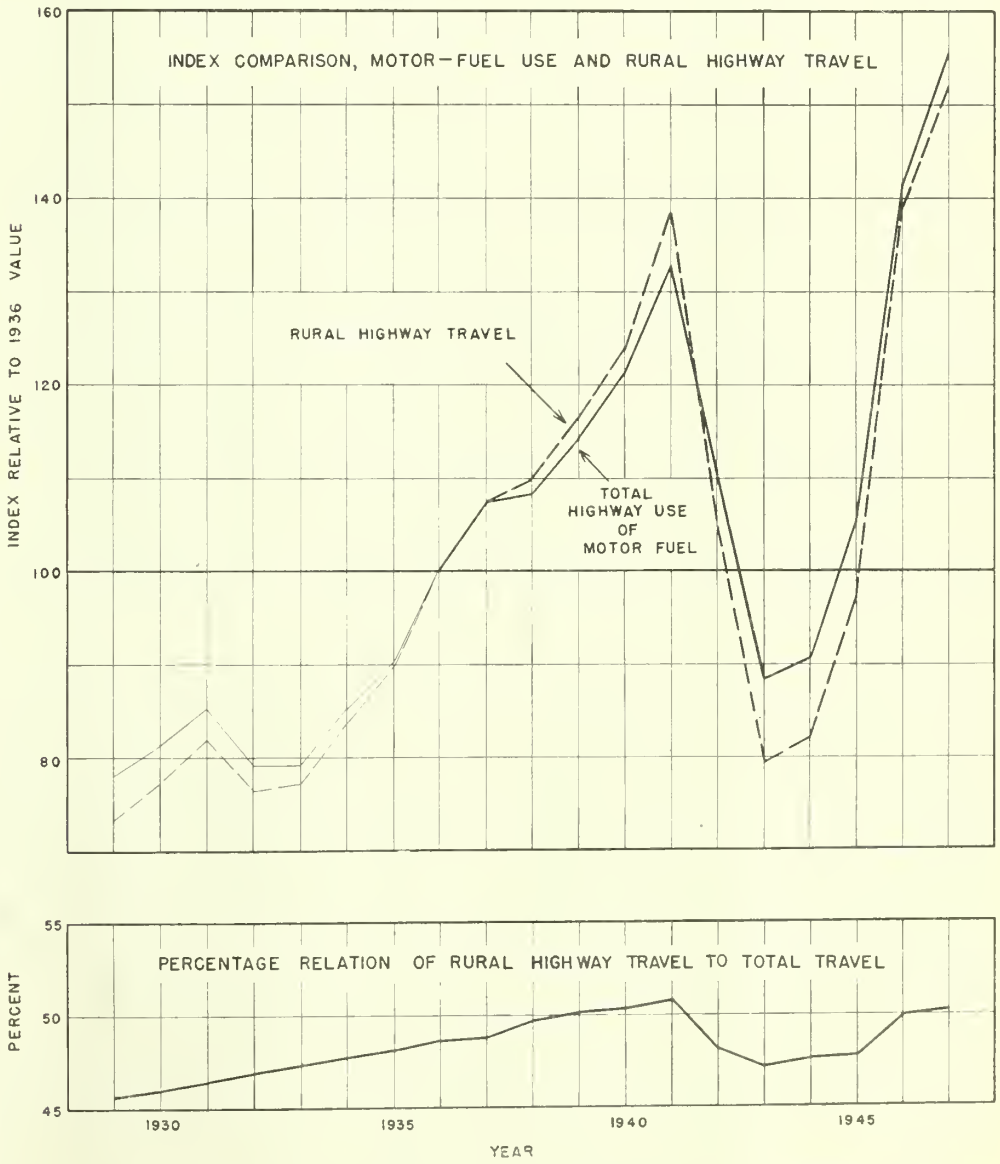


Figure 3.—Trends in total highway use of motor fuel and rural highway travel.

The basic calculations for the annual vehicle-mile estimates, which depended more heavily upon the motor-vehicle-allocation and traffic studies than upon the road-use data, indicated slightly less than 50 percent rural travel in 1936, with the value reaching 50 percent for the first time in 1939.

As may be observed in figure 2, the decline in rural travel during the war period was much more severe than that in urban travel. This situation is reflected in figure 3 by the crossing of the index lines, the motor-fuel consumption index for the years 1942–45 being considerably above the index of rural travel. In the years 1946 and 1947 the recovery of the two indexes has been along more or less parallel lines, but it is shown in the lower panel of figure 3 that the percentage of rural travel, although slightly greater than 50 percent in 1946 and 1947, had not yet quite recovered to its prewar value. Whether it will do so in the next few years is conjectural. Purely from the standpoint of a return to normal trends such a result might be expected. On the other hand, there may have been permanent changes in the relative urbanization of the population which would tend to work against such a development. Furthermore, the current movement toward the relief of urban congestion may have the effect of causing a material increase of urban travel relative to total travel.

MORE ACCURATE AND COMPLETE DATA NEEDED

The making of annual Nation-wide estimates of motor-vehicle travel has proved to be a useful activity, well worth continuing. The value of these estimates would be greatly enhanced if it were possible to subdivide the Nation-wide totals and averages into values for each State. Although vehicle-mile estimates are frequently made in individual States, it has not yet been found feasible to organize a procedure whereby each State highway department would make and report such estimates annually.

Great difficulties attend the calculation of classified vehicle-mile estimates in an individual State. The Nation-wide rural traffic analysis is based primarily on reports from about 660 continuous automatic recorders, an average of about 14 per State. This is a rather slender sample on which to base an estimate of total vehicle-miles, even though reinforced by data from portable recorders, manual counts, and other sources. In calculating averages per vehicle difficulties are encountered in making allowances for the out-of-State travel of vehicles registered in the State and the travel within the State of vehicles from other States. In making use of motor-fuel consumption statistics similar allowances must be made for motor fuel purchased in one State and used in another. These obstacles are not insurmountable, but they introduce sources of inaccuracy to which the calculation of Nation-wide estimates is not subject.

Although a respectable body of data underlies the annual estimate of Nation-wide rural traffic volumes, it is still necessary to predicate the estimates of a given year on the sequence of values obtained for previous years, with the period of the comprehensive traffic surveys taken as the base. In other words, the volume of rural traffic data produced each year is not sufficient to provide independent estimates for that year taken alone. As the base period becomes more and more remote the hazards of this procedure increase. With respect to urban traffic no comparable series of data exists; and Nation-wide estimates of urban travel can be obtained only by the device of estimating total travel with the aid of motor-fuel consumption data, and deducting therefrom the estimated values of rural travel. Increasing recognition of the necessity to know about the characteristics of urban travel gives promise that a body of data on urban traffic, comparable to that now available in the field of rural traffic, will gradually be developed.

It is unlikely, however, that at any time in the near future, continuous records of traffic volumes will be adequate for the making of independent estimates of total vehicle-miles—independent, that is, of the past sequence of such estimates and of such supporting data as the record of motor-fuel consumption. Vehicle-mile estimates are essentially a by-product that would not of itself justify large expenditures in highway research. Studies of traffic volume and composition, origins and destinations, and the patterns of highway travel have as their immediate objectives the solution of problems of highway location, geometric design, and traffic control. More broadly, they are concerned with the adaptation of the highway plant to the travel habits and needs of motor-vehicle users and the public generally; and the coordination of highway planning with land-use and other general planning. Finally, the data collected in this field should be adapted to the requirements of financial planning and the solution of highway tax problems. Estimates such as those described in this report are useful only as they further these main objectives, which do not necessarily require the maintenance of a continuous annual record of large dimensions.

Since it remains necessary to calculate annual estimates of travel volumes in terms of year-to-year changes, referred to a base period of comprehensive studies, it becomes imperative to consider whether a new base period should not be established. We cannot continue much longer to predicate an analysis of postwar patterns of highway use on estimated deviations from patterns found to exist circa 1936. A new deal in comprehensive planning-survey studies is called for.

In the field of rural travel, definite progress is being made in the direction of comprehensive coverage. About 14 States have programs in operation or in the planning stage, which will realize or approximate the scope of the earlier State-wide studies, and thus establish a new base period for rural traffic estimates. In general, these programs call for coverage of the entire rural system over a short period of years, rather than for a single 1-year survey. No such progress with respect to studies of urban travel, or the use of interview or questionnaire techniques to determine the distribution of all travel, can be reported.

There can be no doubt of the general utility, in the solution of current highway problems, of a new supply of "strictly fresh" data on the characteristics of motor-vehicle travel. In the very important studies of highway needs, both engineering and fiscal, that have been made in numerous States during the past few years, the investigators have been forced to rely on an inadequate body of information. A new set of comprehensive surveys would reinforce and give authority to the findings of these studies (although probably necessitating some modifications); and would provide a much better foundation for similar studies in States where they have not yet been made.

Obviously the new studies of highway travel patterns should not follow blindly in the path of the old. Much has been learned about highway research techniques since the days of the early planning surveys. Methods of traffic counting, classification, and truck weighing have become highly developed; and there is continuous study of the efficiency of sampling in this field. The early motor-vehicle-allocation and road-use surveys taught us much about how and how not to sample motor-vehicle travel habits. More recent developments in road-interview and home-interview origin-destination studies have forged new weapons of attack on basic highway problems. The intention here is not to offer a procedure for making comprehensive State-wide studies; but only to point out that the tools of data collection and analysis are ready to our hands. What is needed is careful, intelligent planning, with a shrewd eye on efficiency and economy in sampling, and a sober determination to concentrate on essential facts.

The necessity for using motor-vehicle registration and motor-fuel consumption data in making estimates of highway travel will continue. These data are matters of State record. Although the records themselves can be improved, and efforts are being made in that direction, it is in their utilization that the weakness lies. In order to convert motor-fuel consumption figures into classified estimates of motor-vehicle travel it is necessary to use data on average miles per gallon developed in the early planning-survey studies, with only such corrections as can be devised to account for known changes, such as the increased operating loads of trucks and combinations. In planning comprehensive State-wide studies of motor-vehicle use and travel characteristics, provision should be made for obtaining accurate statistical averages of the rates of fuel consumption in vehicles of different types and loadings.

U. S. GOVERNMENT PRINTING OFFICE: 1949

Statutory Authority of State Highway Departments in Municipalities

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The growth of urban concentrations of traffic has forcefully called attention to the urgent need for highway improvement in the cities of the United States. Urban extensions of the State highway systems frequently are characterized by great physical and functional inadequacies, and are costly to modernize. Nevertheless, the statutory authority of State highway departments in urban areas is still decidedly limited.

Twenty-one States have express powers to finance urban extensions of State primary highways; 16 are specifically authorized to acquire lands for them; 15 States may construct and 20 may maintain such urban routes without restriction. Only six States have unlimited statutory authority in all four fields. In the absence of express authority, the doctrine of implied powers may apply, but the legal status so derived is sometimes precarious.

While the most frequent limitation on State authority in urban areas is placed on size as determined by population, a decided trend is discernible toward removal of these restrictions. Among other limitations, 10 States are restricted in regard to the width and type of highways they can build in cities; and 12 States have varying requirements for local consent before urban extensions of the State highway system can be established.

Even more circumscribed is the statutory authority of the States with respect to urban highways other than extensions of the State primary system. This apparent lack of authority may be offset in part, however, by the power to designate streets in urban areas as extensions of the State system, granted in about half the States.

Almost invariably, in the 28 States where expressways have been sanctioned by law, the authority of the State highway department to establish controlled-access highways in cities is qualified in varying degrees.

More than half the States allocate portions of their highway-user revenues to urban areas, often with the express provision that such funds be used for the improvement of urban highways other than extensions of the State system.

THE CONGESTION that motor travel encounters today in the vicinity of and within the cities and metropolitan areas of the United States constitutes a formidable traffic barrier. These urbanized areas are the sources and destinations of the greater portion, by far, of the heavy flow of traffic that moves over the Nation's highways. Vehicle-mileage of travel on city streets alone constitutes approximately 50 percent of all travel. The principal urban arteries and main commercial streets carry a large percentage of the urban traffic. Because of these ever-increasing demands being made on the urban highway plant, urban accommodations of the State highway system are widely recognized to be grossly inadequate.

Several decades ago, Federal-aid highway legislation and many State highway statutes specifically prohibited the application of funds to urban extensions of the Federal-aid and State highway systems. The then superior need of rural-road improvement made such a prohibition reasonable.

Both Federal and State enactments of the last 5 years, however, have taken cognizance of the need for urban-road improvement. Among other things, the National System of Interstate Highways has been authorized and designated—a system that includes the more important transcity and interregional connections of the State systems. Substantial funds have been specifically earmarked for expenditure in prescribed urban areas.

It was not an objective of this investigation to determine whether present statutory authority of State highway departments in urban areas is commensurate with the present need. Rather, the study seeks to present, as objectively as possible, factual information with respect to legislative authorization for financing, land acquisition, construction, and maintenance of various classes of urban highways under State control.

These consist of urban extensions of the State primary system, of which Federal-aid highways are almost always a part, and of other urban streets, such as those leading to State institutions, or secondary routes. A review of controlled-access highway legislation, insofar as it relates to State highway department authority in cities, is also made.

The investigation is based in part upon an analysis of pertinent sections of the State statutory codes, and in part upon a circularization of the State highway departments. Some of the provisions include administrative practice pursuant to law.

It will be noted in table 1 that, with respect to urban extensions of State highways, the authority of many States may exist only by implication from express powers granted.

URBAN EXTENSIONS OF STATE PRIMARY HIGHWAYS

While State highway departments possess wide powers with respect to the planning, financing, acquisition of lands for, construction, and maintenance of rural State highways, this study reveals that their comparable authority with respect to those portions of State primary routes traversing incorporated areas is much more limited.

Only six States [1] have unrestricted statutory authority to finance, acquire lands for, construct, and maintain urban extensions of State primary highways (see table 1). One of the broadest yet most concise enactments is to be found in Louisiana. The Department of Highways is authorized to include streets and roads in cities, towns, and villages within the State highway system when such streets and roads connect two State highways or form a continuation of any highway within the State highway system: "The highway department shall have full authority to acquire property for, finance, construct, maintain, and operate such highways passing through cities, towns, and villages of the State, together with such

[1] California, Indiana, Louisiana, Massachusetts, New Jersey, and Washington.

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Table 1.—*Summary of statutory authority of State highway departments concerning urban extensions of State primary highways*

State	Authority to finance				Authority to acquire land				Authority to construct				Authority to maintain			
	Yes	No	Qualified	Implied	Yes	No	Qualified	Implied	Yes	No	Qualified	Implied	Yes	No	Qualified	Implied
Alabama			X				X		X				X			
Arizona				X				X				X				X
Arkansas				X				X				X				X
California	X				X				X				X			
Colorado	X				X				X						X	
Connecticut			X					X			X				X	
Delaware	X							X			X		X			
Florida	X							X			X		X			
Georgia	X				X						X				X	
Idaho	X				X				X						X	
Illinois			X		X						X				X	
Indiana	X				X				X				X			
Iowa			X		X						X				X	
Kansas			X				X				X				X	
Kentucky	X						X				X		X			
Louisiana	X				X				X				X			
Maine	X							X				X			X	
Maryland				X				X			X				X	
Massachusetts	X				X				X				X			
Michigan			X				X		X				X			
Minnesota			X					X			X				X	
Mississippi			X				X				X		X			
Missouri	X					X			X				X			
Montana			X			X					X				X	
Nebraska				X				X				X			X	
Nevada				X				X				X				X
New Hampshire			X			X					X				X	
New Jersey	X				X				X				X			
New Mexico				X				X				X				X
New York			X				X				X				X	
North Carolina	X							X	X				X			
North Dakota				X				X			X				X	
Ohio	X							X			X				X	
Oklahoma	X							X	X				X			
Oregon	X				X						X		X			
Pennsylvania			X				X				X				X	
Rhode Island			X		X						X		X			
South Carolina			X		X						X		X			
South Dakota				X				X				X			X	
Tennessee	X				X						X		X			
Texas			X					X			X				X	
Utah	X						X				X		X			
Vermont			X					X			X				X	
Virginia			X		X				X						X	
Washington	X				X				X				X			
West Virginia			X					X			X				X	
Wisconsin	X					X					X				X	
Wyoming			X					X	X						X	
Total	21	0	19	8	16	4	8	20	15	0	26	7	20	0	24	4

off-street parking facilities as may be deemed necessary in connection therewith." [2]

Twenty-one States, and perhaps 8 others by implication, have ample powers with respect to financing urban extensions of State highways; 16 States are specifically authorized to acquire lands for such projects; 15 States, and perhaps 7 others by implication, may construct them; and 20 States, and perhaps 4 others by implication, may maintain without restrictions such urban routes. Implied authority to improve urban extensions of the State highway system is deemed to arise out of the express grant of statutory authority to State highway departments to finance, construct, and maintain State highways. Table 3, pages 166–81, contains a summary of the pertinent legislative provisions.

Urban Federal-aid System

State statutes generally do not deal with urban extensions of Federal-aid primary highways as a separate class of road facilities because practically every mile of such extensions is contained in the more inclusive State primary systems. Accordingly, legal authorization with respect to urban extensions of the State primary systems is deemed to apply to urban extensions of the Federal-aid highways with like force and effect.

Moreover, the States generally have assented to the purposes and provisions of the Federal-aid laws and generally have provided for State cooperation with the Federal Government in effectuating the objectives of the Federal-aid highway program. How much authority of a specific character over urban Federal-aid highways such State legislation grants, apart from the provisions relating to urban extensions of State primary highways, is a matter for conjecture.

It may be helpful to review a few typical enactments of this nature. The Illinois statute is typical of the more generalized of these laws:

> "* * * the General Assembly * * * hereby assents to the provisions, terms, conditions, and purposes of the act of Congress, approved July 11, 1916, entitled 'An act to provide that the United States shall aid the States in construction of rural post roads, and for other purposes,' known as the Federal-Aid Road Act, together with amendments to said act subsequently enacted by the Congress of the United States" (L. 1917, p. 144, sec. 1; Laws 1933, p. 958, sec. 1).

The West Virginia act is more detailed:

> "The legislature of the State of West Virginia hereby renews its assent to the provisions of the act of congress approved July 11, 1916, entitled 'An act to provide that the United States shall aid the States in the construction of rural post roads and for other purposes.' The State road commission is hereby authorized to enter into contracts and agreements with the United States Government relating to the survey, construction, and maintenance of roads under the provisions of said act of Congress, to submit such scheme or program of construction and maintenance as may be required * * * and to do all other things necessary to carry out the cooperation contemplated and provided for by said act * * *" (L. 1921, ch. 112).

The Indiana law is typical of provisions specifically involving the Federal-aid Highway Act of 1944. The State Highway Commission is granted authority:

> "* * * to cooperate with the Federal Government for postwar construction of highways and bridges under the act of Congress, approved December 20, 1944 * * * and to do and perform all acts as in said amendatory act provided, in order to procure for the State of Indiana the proper apportionment of Federal funds available to the State of Indiana for the construction, maintenance, and improvement of highways under the provisions of said amendatory act" (Acts of 1945, ch. 298, sec. 1, p. 1289).

The South Carolina 1945 act authorizes the State Highway Department to cooperate with the Public Roads Administration in carrying out the provisions of the Federal-aid Highway Act of 1944 and amendments thereto, and to do all things necessary to carry out the provisions of said Federal-aid Highway Act, including but not limited to, the planning, construction, and supervision of all eligible projects regardless of whether such projects are a part of the State highway system; but stipulates that if such projects are not on the State highway system local units must match Federal funds. The act further authorizes the State Highway Department and political subdivisions to do all things necessary to carrying out the act, including the acquirement of right-of-way (South Carolina Act 223, Laws 1945, sec. 53).

[2] Louisiana Acts 1942, No. 4, sec. 38; 1944, No. 65, sec. 3.

Financing

More State highway departments possess unqualified authority with respect to the financing of urban extensions of State primary highways than with respect to any other aspect of authority dealt with in this study. Twenty-one jurisdictions are granted such statutory authority.[3] Additionally, similar authority may be implied in eight other States from the statutory grant of general authority with respect to the State highway system, where urban extensions are deemed to be an integral part of that system,[4] or for other reasons indicated in subsequent sections.

The authority is qualified in the remaining 19 States. The nature of the restrictions is indicated as follows:

Alabama: Limited to 50 percent of right-of-way costs.
Connecticut: Limited to one north-south and one east-west route through each city.
Illinois: Limited to type and width required to care for traffic and parking needs.
Iowa: Not more than 25 percent of primary road fund to be spent for this purpose in one year.
Kansas: Limited to type and width of highway outside city.
Michigan: Must be requested by governing body; 100-percent participation only in cities under 20,000 population; 100-percent participation only to width of highway outside city.
Minnesota: Limited to normal width of highway outside city.
Mississippi: Limited to towns of less than 2,500 population.
Montana: Not unless Federal funds are involved.
New Hampshire: Not in compact areas of cities and towns over 2,500 population.
New York: Limited to highways in special categories.
Pennsylvania: Limited to highways in special categories.
Rhode Island: Subject to approval of city.
South Carolina: Limited as to amount of funds which may be spent in cities over 2,500 population.
Texas: Subject to agreement with governing body of municipality.
Vermont: Limited to towns under 1,500 population.
Virginia: Limited to 50 percent of cost in cities over 3,500 population.
West Virginia: Limited to same obligation as for State highways outside municipality.
Wyoming: Limited to 50 percent participation in construction costs in cities over 1,500 population.

As will readily be noted, some of the qualifications on authority are substantial in character, while a few are more apparent than real.

Land Acquisition

One of the most significant limitations on the authority of State highway departments in urban areas concerns the matter of acquiring the necessary rights-of-way. State highway departments in only 16 States have been granted unequivocal authority to acquire lands needed for highway improvements within the corporate limits of cities.[5] In eight other States, the right to acquire highway right-of-way in urbanized areas is qualified in various ways, as follows:

Alabama: Contingent on failure of county or municipality to acquire land; also limited to 50 percent of cost.
Kansas: May contribute to cost in cities of third class.
Kentucky: State may participate in cost of new or additional right-of-way.
Michigan: Subject to municipal consent.
Mississippi: Limited to cities under 2,500 population (presumably) or where houses average more than 200 feet apart for a mile or fraction thereof.
New York: Limited to highways in specified categories.
Pennsylvania: Limited to highways in specified categories.
Utah: Under agreement with municipality.

No authority at the State level is conferred in four States,[6] and in two of them—New Hampshire and Wisconsin—the law specifically reserves the right of acquisition to the local units.

Missouri has been placed in this category because no provision for acquisition of land therefor is included in the 1945 constitution which authorizes the State to finance, construct, and maintain "any highway in any city or town which is found necessary as a continuation of any State or Federal highway, or any connection therewith, into and through such city or town." Prior to enactment of the new constitution, the State's authority to finance, construct, and maintain urban extensions of State highways applied only to such highways in cities under 2,500 population. No authority to acquire land for such highways existed, except by implication; the State having such authority with respect to State highways generally.

Montana appears to have no statutory provision covering the improvement of urban extensions of State highways as such. Although it has been assumed that the State Highway Commission may finance, construct, and maintain such urban extensions if Federal funds are involved, no reference can be found to the acquisition of right-of-way.

Implied authority to acquire land

The laws in the remaining 20 States contain no specific reference to the matter of land acquisition. Whether these States have the authority by implication to acquire land for urban extensions of the State primary system, may be questioned. State statutory authority to construct urban extensions of State highways might conceivably be construed to imply the right to acquire necessary lands therefor, on the assumption that the term "highway" includes a city "street."

The laws of 13 of the 20 States, lacking express authority with respect to acquisition of lands in urban areas, specifically bestow on the State highway department the right to construct urban extensions of State highways and also to acquire lands for State highways generally.[7]

The contention that the remaining seven States[8] have such authority by implication has even less support. In these States the highway department's authority to construct urban extensions of State highways itself can only be said to be by implication, as indicated in the following section.

While the doctrine of implied authority may apply in certain instances to the acquisition of lands for urban highway purposes, its legal status is very precarious. An outstanding legal authority has indicated that it is a recognized rule of construction that one power is implied in the grant of another when the latter cannot be exercised or carried into effect without the exercise of the implied power, since anything within the manifest intention of the makers of a statute is as much within the statute as if it were within the letter of the law. But it is further asserted that there is no power by implication to take private property for public use merely because the final objective cannot be attained without the use of private property. Although the contrary has been held, according to what is held to be the better rule, a statute merely granting power to lay out and establish streets does not authorize the condemnation of lands for such purposes.[9]

The matter of implied authority to acquire lands for urban extensions of State highways, in the absence of express statutory provisions on the matter, was the substance of an attorney general's opinion in Texas several years ago,[10] when the State Highway Department sought to construct a State highway within the boundaries of a home-rule city. Unwilling to acquire the necessary right-of-way, the city consented to have the property condemned by the State Highway Department. The attorney general of Texas ruled that the department lacked authority to do so. Three issues were involved:

Issue: Whether the State Highway Department, or the county, might acquire by purchase or condemnation the right-of-way required in a home-rule city for the construction of a State highway.

Answer: Although the legislature could have transferred jurisdiction of cities and towns over particular highways therein to the State Highway Department, nevertheless, it neither expressed nor implied any intention to do so.

Issue: Whether the city, either by ordinance or agreement with the State, might legally delegate, transfer, or pass on to the State Highway Department or the county, acting for the Department, its authority to condemn such right-of-way.

Answer: Where the legislature delegates the right to exercise the power of eminent domain, the grantee of the power cannot surrender, transfer, or redelegate the same to another unless expressly authorized by the statute conferring the power.

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[3] California, Colorado, Delaware, Florida, Georgia, Idaho, Indiana, Kentucky, Louisiana, Maine, Massachusetts, Missouri, New Jersey, North Carolina, Ohio, Oklahoma, Oregon, Tennessee, Utah, Washington, and Wisconsin.

[4] Arizona, Arkansas, Maryland, Nebraska, Nevada, New Mexico, North Dakota, and South Dakota.

[5] California, Colorado, Georgia, Idaho, Illinois, Indiana, Iowa, Louisiana, Massachusetts, New Jersey, Oregon, Rhode Island, South Carolina, Tennessee, Virginia, and Washington.

[6] Missouri, Montana, New Hampshire, and Wisconsin.

[7] Connecticut, Delaware, Florida, Maryland, Minnesota, North Carolina, North Dakota, Ohio, Oklahoma, Texas, Vermont, West Virginia, and Wyoming. In the case of Ohio, however, the authority extends to the construction of urban extensions of State highways and to the acquisition of lands for enterprises entrusted to its supervision. Such authority is construed to be substantially similar to the other 12 States in this category.

[8] Arizona, Arkansas, Maine, Nebraska, Nevada, New Mexico, and South Dakota.

[9] See *The Law of Municipal Corporations*, by Eugene McQuillin, 2d ed., 1943, rev. vol. 4, sec. 1580, and 1947 cumulative supplement, 2d ed., rev. Case materials are included.

[10] See *Opinions of the Attorney General*, Texas, 1946, Opinion No. 0-7108.

Issue: Whether the Governor of Texas might obtain such right-of-way for the State Highway Department, acting under the provisions of a State statute providing for acquisition of land by the Governor for public use.

Answer: Although the Governor had statutory authority to acquire any land necessary for any public use, the legislature had specifically delegated authority to cities and towns over their streets, which prevailed over the authority of the Governor to acquire land in this case.

A similarly restricted construction of State highway department authority with respect to land acquirement in urban areas is found in other States. Prior to a 1945 enactment in the State of Washington, enlarging the authority of the Department of Highways in urban areas, the attorney general declared the Department had no legal authority to acquire land in its own name within incorporated cities and towns for highway purposes.[11] The "power of condemnation within cities and towns in the State could only arise by construction or by implication * * *. If the power of condemnation does exist, it must be by virtue of specific and clear statutory direction and cannot arise by implication or construction. If construction of a statute is necessary to sustain the power, no such power can exist."[12] Accordingly, it is apparent that State highway department authority with respect to acquisition of land for urban extensions of State highways which exists solely by implication is extremely tenuous.

Construction

The present status of statutory authority of State highway departments in urban areas with respect to financing and land acquisition has now been sketched. The next logical element of authority deals with the right to construct urban extensions of primary State highways. Highway departments in 15 States appear to have the unlimited statutory right to construct such urban facilities.[13]

In 26 other jurisdictions, the right to construct urban roads is restricted in the following manner:

Connecticut: Limited to one north-south and one east-west route through city.
Delaware: May not change width of streets without consent of town.
Florida: Type of construction shall conform to that of State highway.
Georgia: Construction shall comply with all reasonable municipal ordinances, etc.
Illinois: To type and width required to care for traffic and parking needs.
Iowa: Subject to approval of council of municipality; also may not exceed width of primary road system.
Kansas: Type to be determined by agreement between city and State Highway Commission; limited to width and type of highway approaching city.
Kentucky: Subject to approval of city authorities.
Maryland: Must be in conformity with grades approved by municipal authorities.
Minnesota: Only to width outside city; plans and specifications must be approved by municipal authorities.
Mississippi: To width of State highway outside city limits; only in towns of less than 2,500 population.
Montana: Not unless Federal funds are involved.
New Hampshire: Not in compact areas of cities over 2,500 population.
New York: Limited to highways in special categories.
North Dakota: To normal width of State highways.
Ohio: Subject to approval of governing body of municipality if not on Federal-aid system.
Oregon: State may not change or establish grade without consent of governing body of municipality.
Pennsylvania: Limited to highways in special categories.
Rhode Island: Subject to approval of city.
South Carolina: Subject to approval of city in municipalities over 2,500 population.
Tennessee: To width and type State may deem proper but such width shall not be less than 18 feet.
Texas: Under agreement with city.
Utah: Must conform to direction and grade of other streets in such city.
Vermont: Limited to State highways in towns under 1,500 population and supervision of "State-aid connecting links" improved with State funds.
West Virginia: Limited to same obligation as for State highways outside municipalities.
Wisconsin: Construction to be same as for State trunk highways.

In the remaining seven States, the authority to construct urban extensions of the State primary system probably may be implied from a general grant of authority.[14] It is obvious, however, that a judicial interpretation of existing State authority can always upset any presumptions under implied powers and, accordingly, express authority is always to be preferred.

Maintenance

The right to maintain urban extensions of the State primary system is the remaining element of State highway department authority that needs to be considered. The laws of 20 States grant definite authority to the State highway departments with respect to the maintenance of the urban links of the State highway system.[15] In addition, it is possible that similar authority exists by implication in four other jurisdictions.[16]

Authority with respect to maintenance is qualified in the remaining 24 States, in the following manner:

Colorado: Only under agreement with city unless remainder of special 3-percent gas-tax fund is used.
Connecticut: Limited to portions accepted under statutes pertaining to "State aid" or improved at State expense.
Georgia: Must comply with all reasonable municipal ordinances not in conflict with specifications of State Highway Department.
Idaho: Limited to cities under 2,500 population; State may enter into maintenance agreements with cities of 2,500 to 6,000 population.
Illinois: Only to type and width required to care for traffic and parking needs.
Iowa: Subject to approval of city council; limited to width of primary road system.
Kansas: Limited to cities of third class; State pays certain amount per mile to first and second class cities for maintenance.
Maine: Limited to cities under 5,000 population, but may maintain in cities over 5,000 population if county does not do so.
Maryland: Limited to roads constructed by State; excludes Baltimore.
Minnesota: Limited to width outside cities.
Montana: Only if Federal funds are available for construction.
Nebraska: Limited to 100-percent participation in municipalities under 2,500 population and 50-percent participation where population is between 2,500 and 25,000.
New Hampshire: Not in compact areas of cities and towns of over 2,500 population.
New York: Limited to highways in special categories.
North Dakota: Limited to normal width of State highway.
Ohio: Subject to approval of governing body of municipality.
Pennsylvania: Limited to highways in special categories.
South Dakota: Not in cities over 2,500 population.
Texas: Under agreement with city.
Vermont: Limited to cities under 1,500 population.
Virginia: Limited to cities under 3,500 population.
West Virginia: Limited to obligation State is required to assume outside cities.
Wisconsin: Limited to cities under 2,500 population.
Wyoming: Limited to cities under 1,500 population.

Population Limitations

One of the most frequent limitations on State authority in urban areas is placed on size as determined by population; the statutes of 14 States containing such restrictive provisions.[17] In Mississippi and New Hampshire, for example, the State highway departments have no authority in cities of over 2,500 population. In Vermont, State jurisdiction in cities of 1,500 or more inhabitants is restricted to streets where houses average 100 feet or more apart for the space of one-half mile, a provision that had its parallel in the Federal-aid Road Act of 1916 and its amendments.

Some States place restrictions upon the amount of financial aid that may be given to cities in certain population groups. In Michigan, for instance, 100-percent State participation is limited to cities under 20,000 population, with a relative decrease in proportion of State funds to 50 percent in cities of over 50,000 persons. Virginia may provide only 50 percent of the cost of State highway extensions in cities over 3,500, while Wyoming is limited to 50-percent participation in cities over 1,500 population. South Carolina has a comparable provision.

Maintenance provisions and population restrictions are frequently interrelated. State maintenance authority is confined to the smaller urban areas in 10 States.[18] Vermont and Wyoming statutes, to illustrate, limit State maintenance to cities of 1,500 population or under. Idaho, Nebraska, New Hampshire, South Dakota, and Wisconsin may finance 100-percent maintenance costs only in cities under 2,500 population; while in Maine and Virginia, the State highway departments are not responsible for maintenance in cities over 3,500 and 5,000 inhabitants, respectively. The Kansas State Highway Commission pays cities of the first and second class $250 per mile

[11] Based upon an informal opinion of the attorney general rendered August 31, 1944, to the Honorable Burwell Bantz, Director of Highways, State of Washington.

[12] See *State ex rel Chesterley* v. *Superior Court*, 19 Wash. 824-144 Pac. (2d) 916 (1944).

[13] Alabama, California, Colorado, Idaho, Indiana, Louisiana, Massachusetts, Michigan, Missouri, New Jersey, North Carolina, Oklahoma, Virginia, Washington, and Wyoming.

[14] Arizona, Arkansas, Maine, Nebraska, Nevada, New Mexico, and South Dakota.

[15] Alabama, California, Delaware, Florida, Indiana, Kentucky, Louisiana, Massachusetts, Michigan, Mississippi, Missouri, New Jersey, North Carolina, Oklahoma, Oregon, Rhode Island, South Carolina, Tennessee, Utah, and Washington.

[16] Arizona, Arkansas, Nevada, and New Mexico.

[17] Idaho, Kansas, Maine, Michigan, Mississippi, Nebraska, New Hampshire, New York, South Carolina, South Dakota, Vermont, Virginia, Wisconsin, and Wyoming.

[18] Idaho, Kansas, Maine, Nebraska, New Hampshire, South Dakota, Vermont, Virginia, Wisconsin, and Wyoming.

for maintenance, while cities of the third class may have their State highway routes maintained by the State or be paid $250 per mile for doing the work themselves.

A decided trend toward the removal of these State statutory population restrictions is now discernible.[19] While some States began removing these limitations in the early 1930's, more recent illustrations are abundant. For example, the Colorado Legislature amended the State highway law in 1945, eliminating a provision restricting State assistance in financing and constructing State highway extensions to those within incorporated cities or towns having a population of less than 2,500. In the same year, Indiana removed a statutory limitation on State Highway Commission authority in cities of the first class, thereby making State road improvements possible in Indianapolis. Kentucky, Missouri, Rhode Island, South Carolina, Virginia, Wyoming, and Wisconsin provide other recent illustrations.

Type and Width Restrictions

Ten State statutes [20] contain restrictions relating to the width and type of highway that State highway departments are authorized to establish in urban areas. Iowa, Minnesota, and North Dakota laws, for example, limit the width which the State may construct in urban areas to that prevailing outside the corporate limits of the city or town.

In Minnesota and North Dakota, however, the State highway departments are authorized to enter into agreement with the municipality concerning the establishment of a greater width than that which obtains outside the municipality, the extra cost to be absorbed by the local unit. A similar provision in West Virginia merely indicates that the city or town must pay the extra expense if the cost of the portion of the road within the urban area exceeds that outside the corporate limits for comparable facilities. In Michigan, the State Highway Department may participate to the extent of 50 percent of the cost of any additional width.

The law in Florida provides that the type of construction within urban areas shall conform to that used on State rural roads generally, while the Kansas statute limits the width and type of improvement within municipalities to that outside.

Table 2.—*The relationship of urban mileages under State control to primary systems and total urban mileages*

State	Urban mileage under State control			Mileage of State primary system	Total urban mileage
	Miles	Percentage of State primary system	Percentage of total urban mileage		
Alabama	597	8.1	13.9	7,371	4,306
Arizona	67	1.7	5.1	3,864	1,324
Arkansas	440	4.5	9.5	9,754	4,609
California	1,111	21.7	6.5	5,124	17,218
Colorado	411	9.9	11.6	4,170	3,537
Connecticut	399	13.7	10.3	2,913	3,862
Delaware	103	9.2	19.5	1,118	[1] 528
Florida	987	11.3	8.3	8,737	11,908
Georgia	1,418	9.9	26.3	14,389	5,396
Idaho	204	3.9	13.3	5,170	1,536
Illinois	1,658	13.8	7.4	11,989	22,391
Indiana	876	8.4	8.0	10,422	10,990
Iowa	1,052	10.8	8.0	9,717	13,144
Kansas	506	5.1	7.0	9,896	[1] 7,214
Kentucky	463	4.4	12.9	10,532	3,585
Louisiana	760	16.7	21.0	4,561	3,688
Maine	397	12.4	36.8	3,198	[1] 1,080
Maryland	93	2.1	3.7	4,521	2,518
Massachusetts	120	6.2	1.9	1,940	6,226
Michigan	1,038	10.9	7.7	9,522	13,526
Minnesota	1,294	11.5	11.5	11,221	11,260
Mississippi	353	8.9	9.1	3,954	3,882
Missouri	731	8.5	5.8	8,612	12,703
Montana	194	3.3	8.8	5,807	2,200
Nebraska	405	4.4	8.0	9,220	5,056
Nevada	66	3.0	15.8	2,224	418
New Hampshire	77	4.8	7.5	1,590	1,024
New Jersey	378	22.3	4.1	1,697	9,161
New Mexico	230	2.3	18.2	10,089	1,266
New York [2]				14,225	[1] 17,358
North Carolina	1,032	9.1	17.4	11,362	[1] 5,932
North Dakota	199	2.9	23.0	6,963	865
Ohio	2,323	12.6	12.0	18,390	19,318
Oklahoma	500	4.9	6.5	10,104	7,644
Oregon	355	7.4	7.9	4,822	4,467
Pennsylvania	2,867	18.2	17.6	15,747	16,312
Rhode Island	101	12.2	7.0	831	1,446
South Carolina	1,063	12.8	36.1	8,327	2,947
South Dakota	170	2.8	7.1	6,030	2,381
Tennessee	511	6.7	14.9	7,576	3,424
Texas	1,550	5.5	7.2	28,377	21,425
Utah	558	10.5	17.7	5,302	3,151
Vermont	55	3.0	7.0	1,804	787
Virginia	277	3.0	6.9	9,324	[1] 3,999
Washington	306	7.4	4.6	4,139	6,583
West Virginia	399	8.2	18.2	4,884	2,194
Wisconsin	806	7.6	9.5	10,657	8,484
Wyoming	48	1.1	5.5	4,382	875
Totals	29,548	8.1	9.4	366,568	315,148

[1] Estimated.
[2] No urban extensions under State control.

Local Consent Requirements

The enactments of 12 States [21] contain varying requirements for local consent before the State highway department may establish urban extensions of the State system in urban areas. In Iowa, Kentucky, and Rhode Island, the laws make State improvements on urban extensions contingent on the approval of the governing body of the municipality involved. The Ohio code contains a similar provision, excepting from the consent provision, however, any improvement of a State highway in a municipality if it constitutes a part of the Federal-aid highway system.

Not quite as broad a restriction is the Delaware requirement that the State may not change the width of a street without the consent of the town traversed by the highway. Street construction by the State in Maryland and Oregon must be in conformity with grades approved by municipal authorities. Plans and specifications in Minnesota must be approved by the municipality. Georgia law indicates that construction shall comply with all reasonable municipal ordinances. The type of improvement undertaken by the State in Kansas is to be determined by agreement between the city and the State highway authorities. Construction of State highway extensions in Utah, undertaken by agreement with local officials, shall conform to the direction and grade of other streets in the municipality.

Designation of Urban Extensions

The power to designate streets in urban areas as additional extensions of the State highway system can provide the basis for improvement of urban highways other than present extensions of the State system.

(Text continued on page 182)

[19] A counterpart of this trend also exists with respect to Federal-aid legislation. The Federal-aid Road Act of 1916 contained a restriction on the application of Federal funds that excluded "every street and road in a place having a population, as shown by the latest available Federal census, of two thousand five hundred or more, except that portion of any such street or road along which the houses average more than two hundred feet apart" (sec. 2). This section was repealed by the Hayden-Cartwright Act of 1934 (sec. 13). The objective of authorizing highway expenditures in urban areas under the Federal emergency construction program was based primarily upon maximum employment considerations. Aside from the defense highway program initiated in 1941 by the Defense Highway Act, the needs of urban transportation as such were first implemented on an organized basis in the Federal-aid Highway Act of 1944.

[20] Florida, Illinois, Iowa, Kansas, Michigan, Minnesota, Mississippi, North Dakota, Tennessee, and West Virginia.

[21] Delaware, Georgia, Iowa, Kansas, Kentucky, Maryland, Minnesota, Ohio, Oregon, Rhode Island, South Carolina, and Utah.

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
ALABAMA	YES But State may finance only 50 percent of cost of right-of-way.	YES But right is contingent upon failure by county or municipality to acquire lands.	YES Provided Director of Highways has designated the streets as connecting links.	YES Provided Director of Highways has designated the streets as connecting links.
ARIZONA	YES By implication since State has this authority for State highways and may, by agreement with incorporated cities or towns, designate streets as State highways.	YES By implication since State has this authority for State highways and may, by agreement with incorporated cities or towns, designate streets as State highways.	YES By implication since State has this authority for State highways and may, by agreement with incorporated cities or towns, designate streets as State highways.	YES By implication since State has this authority for State highways and may, by agreement with incorporated cities or towns, designate streets as State highways.
ARKANSAS	YES By implication since State is authorized to use any funds available for construction and maintenance of roads embraced in State highway system, which extends through cities.	YES By implication since State has power of eminent domain for carrying out enterprises entrusted to its supervision. Only an easement for highway purposes may be acquired.	YES By implication since State is authorized to construct roads in State highway system, which extends through cities. Construction work to be distributed throughout counties so as to maintain a parity between them as far as practical.	YES By implication since State is authorized to maintain roads in State highway system, which extends through cities.
CALIFORNIA	YES State highways in cities shall be maintained, constructed, and improved out of moneys in State highway fund.	YES State may acquire any real property which it considers necessary for State highways including those within cities, or may request governing body of city to acquire necessary land—title or interest to be taken in name of State.	YES Or may enter into cooperative agreement with city for performance of work.	YES Or may enter into cooperative agreement with city for performance of work.
COLORADO	YES State highways include streets forming necessary or convenient links for carrying such highways through cities and towns. In addition to other funds, remainder of 3-percent motor-fuel-tax fund, created by act of 1935, to be used for construction and maintenance of streets, roads, or highways on designated State highway system, in proportion to motor-vehicle registration.	YES	YES State highways may be designated, established, and constructed in, into, or through cities and towns when such highways form necessary or convenient connecting links for carrying State highways through cities and towns.	NO Unless funds are derived from remainder of 3-percent motor-fuel-tax fund allocated for improvement of State highways in towns, cities, and counties under 1935 act, State highways in cities or towns to be maintained by city or town, except that by agreement between State and city or town, State may perform or pay for all or part of maintenance.
CONNECTICUT	YES But only one east-west route and one north-south route through each city and town shall be included in trunk line highway system.	YES By implication since State may acquire land for lay-out, construction, repair, or maintenance of any trunk line highways, which may extend through cities.	YES But only one east-west route and one north-south route through each city and town shall be included in trunk line highway system.	YES Except those portions of trunk line system not built or not accepted under statutes relating to State aid or not improved at State expense.

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
No Except on streets abutting a State educational or charitable institution, or through its grounds. In such case State participation may vary from 50 to 100 percent. State also allocates portion of motor-vehicle and fuel-tax funds to cities and towns for construction, improvement, and maintenance of highways and streets.	No Except on streets abutting a State educational or charitable institution or through its grounds.	No Except on streets abutting a State educational or charitable institution or through its grounds.	No Except on streets abutting a State educational or charitable institution or through its grounds.		A 1945 act provides for cooperation with the United States to obtain benefits of Federal-aid Highway Act of 1944. State Highway Department has power to designate State highways.
No Except by implicatiou on State routes designated as potential State highways.	No Except by implication on State routes designated as potential State highways.	No Except by implication on State routes designated as potential State highways.	No Except by implication on State routes designated as potential State highways.		State Highway Commission is authorized to lay out and establish a complete system of State routes and to determine what State routes or portions thereof shall be accepted as State highways. Under designated conditions Commission may designate secondary State highways. State Highway Commission and governing body of cities may select routes of State highways through cities. By administrative act, 10 percent of State highway fund (i. e., one-third of 30 percent allocated to counties) is distributed to cities for improvement, construction, and maintenance of municipal streets and highways in proportion to city population in each county.
No	No	No But State may perform engineering service and supervision of construction on the basis of 100 percent reimbursement by the organization financing the work.	No		State highways are defined by statute to be those primary, secondary, and connecting roads as shown on an official "Map of Arkansas showing State Highway System," including those portions of said routes extending into or through incorporated towns and cities. The State Highway Commission may make such changes in State highway system as it deems necessary but may not eliminate any part of system. Provision is made for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1916.
Yes Five-eighths of 1 cent per gallon tax allocated to cities on basis of population; ⅜ for construction of streets included in system of major streets; ⅖ for maintenance of system of major streets and of secondary street system.	No Unless under act of 1945 authorizing agreements between State and any city for acquisition of property for, construction, improvement, and maintenance of any highway including those not on State highway system to be constructed with Federal aid.	Yes But the State shall delegate such functions to city if satisfied that city is equipped to conduct work efficiently.	Yes But the State shall delegate such functions to city if satisfied that city is equipped to conduct work efficiently.	Freeways are identified with State highways. State is authorized to enter into agreements with any city council concerning the closing of any city street at or near the point of its intersection with any freeway or to make provision for carrying such city street over or under or to a connection with the freeway. The State may do any work on such city street as is necessary.	State highways are designated by constitution or legislature. Act of 1945 provides for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1944.
No But State allocates portion of motor-fuel-tax funds to cities and incorporated towns through counties, on basis of number of motor-vehicle licenses issued, for work on public highways and streets within corporate limits.	No	No	No	Freeways are identified with State highways. State highway engineer, with approval of governor, is authorized to enter into agreements with cities or towns having jurisdiction over city or town streets concerning the closing of any city street at or near point of its intersection with the freeway or to make provisions for carrying such city street over or under or to a connection with the freeway and shall do any and all work on such city street as is necessary therefor. Authority extends to establishment of local service roads in connection with freeways.	1945 act authorizes funds for cooperation with Federal Government to obtain benefits of Federal-aid Act of 1944. State highways may be designated into or through cities when such highways form necessary or convenient connecting links for carrying State highways into or through such cities.
Yes In connection with State-aid highways on mileage basis. State allocates $5 million per year to towns for improvement or maintenance of local roads or streets and $1 million for improvement of local dirt and unimproved roads on mileage basis.	Yes In connection with State-aid highways.	Yes In connection with State-aid highways.	Yes In connection with State-aid highways.	Parkways and freeways are identified with State trunk line system. State is authorized to lay out and construct service highways to provide access from areas adjacent to parkway or freeway.	Trunk line system is established by law but State Highway Commissioner may relocate any section for safety purposes. Provision is made for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1916.

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
DELAWARE	YES Construction and maintenance shall be at sole expense of State unless governing body of incorporated town agrees to other terms.	YES By implication since State may acquire land for State highways, which extend through cities.	YES And State has exclusive control over roads so built, except that State may not change width of highway in incorporated town except with consent of town.	YES State has sole control over maintenance. Urban extensions of State highways shall not be undermined, broken, opened, or torn for any purpose except by written permit of State.
FLORIDA	YES It being expressly declared by the legislature that city and town streets and roads, viaducts, and bridges that constitute connections between or extensions of State roads as municipal connecting links serve a State purpose and are for general benefit of State; that it is a legitimate State function to provide cost of maintenance, repair, construction, and reconstruction.	YES By implication since State may acquire land for State highways and certain streets may be designated as municipal connecting links of State highway system.	YES The State shall construct or reconstruct municipal links to conform to type of construction used on State roads. State is not obliged to remove or re-lay any underground sanitary sewers or gas or water mains or other public utility.	YES State is authorized and directed to maintain and repair designated municipal connecting links. State is authorized to enter into any contract with cities and towns and Federal Government for such purposes. State not required to sweep, sprinkle, or light municipal connecting link roads. Permit for public utility installations must be obtained from State with concurrence of affected city or town.
GEORGIA	YES With respect to streets or portions thereof forming continuations of or links in State-aid system of roads.	YES With respect to streets or portions thereof forming continuations of or links in State-aid system of roads.	YES With respect to streets or portions thereof forming continuations of or links in State-aid system of roads. State construction shall comply with all reasonable municipal ordinances not in conflict with specifications, rules, and regulations of State Highway Commission which acts as advisory board to State Highway Director.	YES With respect to streets or portions thereof forming continuations of or links in State-aid system of roads. State maintenance shall comply with all reasonable municipal ordinances not in conflict with specifications of State. This shall not be construed as relieving the several municipalities of maintaining their streets in a reasonably safe condition.
IDAHO	YES And apportionment of cost of joint improvement or construction as between State and counties, highway or good roads districts, cities, or villages shall be fixed by agreement.	YES Right-of-way for any State highway to be constructed by cooperation of State with any city or village may be procured by State, county, highway district, city, or village, jointly or severally.	YES State has exclusive jurisdiction, supervision, and control over location, maintenance, repair, and reconstruction of street within corporate limits of city, village, county, or highway district, designated as part of a State highway. State authorized to enter into agreements with counties, highway or good road districts, cities, and incorporated villages for improvement of existing State highways.	NO Unless population is under 2,500 or if such road is part of a city or incorporated village having population of more than 2,500 and less than 6,000, State may enter into maintenance agreement with such city or incorporated village. Apportionment of cost in such case is not to exceed 50 percent to be paid by State. In cities of 6,000 population or more, maintenance shall be wholly at expense of city.
ILLINOIS	YES But only to type and width required to care for traffic and parking needs.	YES If designated by Department of Public Works as street or thoroughfare to form part of or connect with State highway system leading up to corporate limit of city.	YES But only to type and width required to care for traffic and parking needs.	YES But only to type and width required to care for traffic and parking needs.
INDIANA	YES Shall maintain and, to extent funds are available, may construct and improve roadway of streets or any part thereof, to such width as may be determined by State in all incorporated cities and towns over which highways in State highway system are routed.	YES But may allow or require cities or towns affected to procure at own expense.	YES May construct and improve streets or any part thereof to such width as may be determined by State in all incorporated cities and towns over which highways in State highway system are routed.	YES Shall maintain streets over which State highways are routed, including curbs and gutters, catch basins, and inlets. May regulate and control parking on streets.

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
No Except with respect to roads leading from any public road to any institution owned or controlled in whole or in part by State.	No Except with respect to roads leading from any public road to any institution owned or controlled in whole or in part by State.	No Except with respect to roads leading from any public road to any institution owned or controlled in whole or in part by State.	No Except with respect to roads leading from any public road to any institution owned or controlled in whole or in part by State.		State Highway Department designates State highway system.
No	No	No	No	Limited-access highways are identified with State highways. State highway authorities may establish limited-access facilities and local service roads wherever they are of opinion that traffic conditions warrant, subject to municipal consent within incorporated cities and towns, and are authorized to enter into agreement with cities, towns, or villages respecting financing, planning, establishment, improvement, maintenance, use, regulation, or vacation of such facilities.	Legislature designates State highway system but road department may determine and fix lines and locations between cities. State road department designates municipal connecting link roads. A 1945 act provides for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1944.
YES State has authority to cooperate with Federal Government, counties, and municipalities in planning, supervising, locating, improving, and constructing roads and streets.	No	YES State has authority to cooperate with Federal Government, counties, and municipalities in planning, supervising, locating, improving, and constructing roads and streets.	YES State has authority to cooperate with Federal Government, counties, and municipalities in planning, supervising, locating, improving, and constructing roads and streets.		State Highway Board designates State highways, including streets, as constituting part of State-aid system, with proper notice thereof to local chief executive.
No But State may cooperate with Federal Government, counties, highway districts, and municipalities for construction, improvement, and maintenance of secondary or feeder roads not on State highway system.	No	No But State may cooperate with Federal Government, counties, highway districts, and municipalities for construction, improvement, and maintenance of secondary or feeder roads not on State highway system.	No But State may cooperate with Federal Government, counties, highway districts, and municipalities for construction, improvement, and maintenance of secondary or feeder roads not on State highway system.		Department of Public Works designates State highways, including highways within limits of incorporated cities and villages. A 1945 act provides for cooperation with the United States under any act of Congress. City streets are specifically included.
YES State allocates part of motor-fuel-tax fund to cities for construction and maintenance of system of arterial highways, and other improved streets other than State highways, subject to consent of State, and apportioned in accordance with population as determined by last census.	No	No But work done with funds allocated from motor-fuel-tax fund is subject to approval of State.	No	Department of Public Works is authorized to designate and establish any existing or proposed highway under its jurisdiction as a freeway; has same authority to limit access, acquire land, and establish local service roads as for State highways; may by agreement with city or village provide for elimination of highway crossings at grade.	State highway system designated by legislature but Department of Public Works may relocate urban extensions to better serve traffic demands. Provision is made for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1916 and subsequent amendments enacted by Congress. State highways include: (1) State-aid roads, (2) State bond issue roads, (3) Federal-aid roads, (4) Highways constructed by State in cities, villages, and towns on routes designated as 1 to 46 of State bond issue act of 1917, (5) State highway extensions in cities, towns, and villages, (6) Streets in cities of 7,500 inhabitants or less where State made refunds to cities for construction, (7) Belt line routes through or around cities to connect State highways, (8) All highways subsequently added to and included within such system.
YES State allocates part of motor-vehicle and motor-fuel-tax funds to cities and towns for construction and maintenance of streets and alleys on basis of population. State may match funds apportioned by Congress for construction of secondary or feeder roads through municipalities with consent of municipality and county.	No	No But may construct secondary or feeder roads through municipalities with matched Federal funds subject to consent of municipality and county.	No But municipality or county is authorized to enter into agreement with State for maintenance of secondary or feeder roads through municipality, if improved with Federal and State funds.	State highway authorities acting alone or in cooperation with local agencies are authorized to participate in establishment of limited-access facilities and local service roads, provided that within cities and towns such authority shall be subject to such municipal consent as may be provided by law. State may enter into agreements with cities and towns respecting financing, planning, establishing, improvement, maintenance, use, regulation, or vacation of limited-access facilities under their jurisdiction.	State Highway Commission may add routes to primary system subject to approval of governor. Provision is made for cooperation with Federal Government to secure benefits of present and future Federal allotments.

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
IOWA	YES Provided not more than 25 percent of primary road fund is used for this purpose in one year.	YES May purchase or condemn necessary right-of-way for extension of primary roads within cities.	YES Subject to approval of council, may construct extensions of primary road system within any city or town, including cities under special charter, but width shall not exceed that of primary road system.	YES Subject to approval of council, may maintain extension of primary road system within any city or town including cities under special charter. Cities over 2,500 may elect to maintain extensions in which case State pays $350 per year per mile.
KANSAS	YES On connecting links of State highway system to same width and comparable type as that of State highway approaching city. State allocates $250 per mile per year from highway fund for maintenance of connecting links of State highway system through cities.	NO But State may contribute to cost of acquisition in cities of third class.	YES May assist cities in construction and reconstruction of such streets as are connecting links on State highway system, to width and type of highway approaching city. Type of construction to be determined by agreement between city and State.	NO But in cities of first and second classes, State is required to pay city $250 per mile per year for maintenance of connections on State highway system; may maintain in cities of third class or pay city $250 per mile therefor.
KENTUCKY	YES On connecting links of State or Federal-aid highways or such city streets as State determines to be feeder roads necessary to produce adequate system of highways.	NO City must deed right-of-way it owns to State. State may participate in new or additional right-of-way.	YES When such streets have been designated by the State as connecting links of State primary roads or feeder streets necessary to produce adequate system of highways. Type to be as high as that outside city. Plans and specifications subject to approval of city authorities and city must establish grade in conformity therewith.	YES On connecting links of Federal-aid or State highways, or on city streets or portions thereof over which such highways are routed, or on feeder streets necessary to produce adequate system of highways, but State need not sweep, sprinkle, light, or police same.
LOUISIANA	YES On streets and roads in cities, towns, and villages, which connect two State highways or form continuations of any State highway, State has full authority to finance. Maintenance may be limited to 50 percent participation if population is 10,000 or over.	YES State has full authority to acquire right-of-way for streets and roads in cities, towns, and villages, which connect two State highways or form continuations of any State highway.	YES State has full authority to construct streets and roads in cities, towns, and villages, which connect two State highways or form continuations of any State highway.	YES State has full authority to maintain streets and roads in cities, towns, and villages, which connect two State highways or form continuations of any State highway.
MAINE	YES Expense of constructing State highways shall be borne wholly by State, except as otherwise provided. No funds for construction derived from any bond issue to be expended on highways within compact portions of towns of over 5,000 population.	YES By implication since State may purchase, take over, and hold for public use such land as may be necessary for State highways, which extend through built-up or compact portions of towns.	YES By implication since State is authorized to construct and maintain State highways, which extend through built-up or compact portions of towns.	NO Except in cities under 5,000 population and in cities over 5,000 population where houses are more than 200 feet apart for distance of ¼ mile; if city fails to maintain, State may perform work and collect from town.
MARYLAND	YES By implication since State highways are presumed to continue through cities. Specific authority exists to finance State highways extending through Baltimore.	YES By implication since State has such authority for State highways, which are presumed to continue through cities.	YES But streets or highways in incorporated cities must be in conformity with street grades approved by municipal authorities.	YES With respect to all State roads constructed by State or brought under its control, but roads improved by State in Baltimore become city streets.

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
YES If Federal funds are allocated for improvement of streets and highways with provision that work shall be under supervision of State. State funds may also be used on continuations of farm-to-market system on roads or streets in any town and in cities under 2,500 population or where houses average not less than 200 feet apart. State also allocates ⅔ of 1 cent tax for construction, reconstruction, repair, and maintenance of roads and streets in incorporated cities and towns in proportion to population.	NO Except for relocation, establishment, or improvement of farm-to-market roads in cities and towns under 2,500 population or where houses average not less than 200 feet apart.	NO Unless Federal funds are allotted for improvement of streets and highways with provision that work be under supervision of State. Improvement of farm-to-market system in towns or cities under 2,500 population or where houses average not less than 200 feet apart is subject to approval of State.	NO Except by contractual agreement with municipal authorities.		State highway commission may add roads to primary system to afford access to cities, towns, or State parks, to shorten direct line of travel on important routes, or to effect connections with interstate roads at State line. Provision is made for cooperation with Federal Government to secure benefits of present or future Federal allotments.
YES If Federal or State funds are used on streets designated by city as major traffic streets in counties having population of over 120,000 and assessed valuation of over $150 million; also on city streets designated as work relief, public employment, or public works projects; may also expend Federal funds for road improvements in "Benefit Districts," under certain conditions.	NO	NO Except in so-called benefit districts, under certain conditions.	NO		State highway commission may designate State highways and certain streets as connecting links of State highway system. A 1945 act provides for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1944.
YES With respect to such city streets as State determines to be feeder roads necessary to produce adequate system of highways.	NO	YES With respect to such city streets as State determines to be feeder roads necessary to produce adequate system of highways. Plans subject to approval of city.	YES With respect to such city streets as State determines to be feeder roads necessary to produce adequate system of highways. Sweeping, sprinkling, lighting, or policing are excluded.	State, acting alone or in cooperation with local agencies, authorized to participate in establishment of limited-access facilities and local service roads, provided that within limits of cities and villages such authority shall be subject to such municipal consent as may be provided by law. State and highway authorities of cities, towns, and villages are authorized to enter into agreements respecting financing, planning, establishment, improvement, maintenance, use, regulation, or vacation of limited-access facilities in their respective jurisdictions.	Department of highways responsible for designation of State primary system of highways and connecting links in cities. Provision is made for cooperation with Federal Government to obtain benefits of any act of Congress.
YES State may finance spur or stub connections from State highways to official principal business district in cities, towns, or villages where no such connections exist. State allocates 1 cent of special 2-cent gasoline tax fund for construction and maintenance of roads and bridges in the parishes and streets and bridges in the city of New Orleans.	YES State may acquire property for spur or stub connections from State highways to official principal business districts in cities, towns, or villages where no such connections exist.	YES State may construct spur or stub connections from State highways to official principal business districts in cities, towns, or villages where no such connections exist.	YES State may maintain spur or stub connections built by State to connect State highways with official business districts in cities, towns, or villages.	Limited-access highways identified with State highways. State is authorized to establish limited-access facilities and local service roads, subject to such municipal consent as may be provided by law within cities and villages. State also authorized to enter into agreements with governing authorities of cities, towns, and villages respecting financing, planning, establishment, improvement, maintenance, use, regulation, or vacation of limited-access facilities in their respective jurisdictions. State may also provide for elimination of intersections at grade of limited-access facilities with existing city, town, or village street.	Department of highways, subject to approval of State highway board, may add routes to State highway system and may include streets and roads in cities, towns, and villages forming continuations of State highways or connecting two State highways. State may enter into any agreements with Federal Government for purpose of improving public highways.
YES State apportions funds to be matched by municipalities in varying proportions for improvement of State-aid roads. State appropriates money to municipalities by special resolve for road improvements. State town improvement fund apportioned to municipalities on basis of mileage of unimproved roads.	YES For State-aid roads.	YES Construction of State-aid roads is under supervision and according to specifications of State. Money appropriated to cities by special resolve to be spent under supervision of State. Town improvement funds to be spent under supervision of State.	YES State-aid highways to be contiuually maintained under direction and control of State at joint expense of State and municipality. Money appropriated to cities by special resolve to be spent under supervision of State. Roads improved with town improvement funds to be maintained by municipality.	Parkways and freeways identified with State highways. State highway commission, under authorization of Governor and council, may lay out and construct parkways or freeways and service roads, provide for restriction or elimination of cross traffic, and acquire land in same manner as for State highways.	State highway system extends through built-up or compact portions of towns. State-aid roads have been designated within compact and built-up sections of cities and towns. Third-class highways may extend through municipalities (these are to be absorbed in State-aid system). State highway commission designates State highways.
YES State allocates funds to counties and city of Baltimore for improvement of streets of incorporated towns, municipalities, etc. Special allocations are made to cities and towns in certain counties. State may finance "motorways" from tolls or other available funds.	NO Except State may acquire lands for "motorways" to be financed from tolls or other available funds.	NO Except State has special authority to construct "motorways" to be finauced from tolls or other available funds.	YES State has special authority to maiutain "motorways." State is also authorized to maintain county roads or streets of incorporated towns or municipalities in certain counties.	Parkways and freeways identified with State highways. State may lay out, establish, and construct any State highway as a parkway or freeway; may acquire property in same manner as for State highways; may regulate, restrict, or prohibit access from any existing or new street.	State Roads Commission has authority to select general system of State roads and highways. Provision is made for cooperatiou with Federal Government to obtain beuefits under Federal-aid Highway Act of 1944.

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
MASSACHUSETTS	YES All State highways are constructed by State wherever located.	YES May take land for State highways or connecting ways.	YES State has authority to construct State highways wherever convenient or necessary.	YES State may maintain all State highways or contract with town to do work.
MICHIGAN	YES On urban extensions of trunk line highways or connecting routes if requested by legislative body, 100 percent if population under 20,000 to 50 percent if population over 50,000; also 50 percent of cost of additional width over that outside city limits.	YES On all trunk highways and connecting links subject to municipal consent.	YES Or may contract with cities or villages to do work.	YES Or may contract with cities or villages to do work.
MINNESOTA	YES To normal width of highway outside city.	YES By implication since State is authorized to acquire by purchase, gift, or condemnation all necessary right-of-way for laying out and constructing trunk highway system, which extends through cities.	YES To width outside city, but plans and specifications must be approved by municipal authorities. Cities may perform work under agreement with State.	YES To width outside city, but city may do work under agreement with State.
MISSISSIPPI	NO Except in towns of less than 2,500 inhabitants or where houses average more than 200 feet apart "for a mile or fraction thereof."	NO Except in towns of less than 2,500 population or where houses average more than 200 feet apart "for a mile or fraction thereof," wherein State may acquire right-of-way of such width as it may determine to be necessary.	NO Except in towns of less than 2,500 population or where houses average more than 200 feet apart "for a mile or fraction thereof." State may construct to width of State highway outside town.	YES Whenever streets of municipality are used by State as part of its primary road system.
MISSOURI (Based largely on the 1945 State constitution.)	YES Any highway in any city or town found necessary as continuation of any State or Federal highway or any connection therewith into and through city or town.	NO	YES On continuations of State highways or connections therewith into and through cities and towns without limitations concerning width or type.	YES But may enter into contracts with cities, counties, or other political subdivisions to do work.
MONTANA	NO Unless Federal funds are involved	NO	NO Unless Federal funds are involved	NO Unless Federal funds are involved
NEBRASKA	YES By implication since State is responsible for improvement and maintenance of State highway system, which apparently runs through cities. May finance 100-percent of maintenance costs only in cities under 2,500 population and 50-percent in cities between 2,500 and 25,000 population, to 24-foot width.	YES By implication since State may acquire land for State highways, which apparently extend through cities.	YES By implication since State is responsible for improvement of State highway system, which apparently extends through cities.	YES State is required to maintain city streets in municipalities under 2,500 population. In municipalities having population of more than 2,500 and less than 25,000, State must maintain to extent of one-half cost and not to exceed 24-foot width.

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
YES State may match Federal funds for improvement of highways in cities or towns subject to consent of mayor or selectmen. State may allocate funds for expenditure under its direction for maintaining, repairing, improving, and constructing town and county highways. "Town" includes city. State may also allocate funds for expenditure under direction of Metropolitan District Commission for maintenance of boulevards in Boston Metropolitan Parks District and for State's share of cost of construction of boulevards within said District.	NO Except State may obtain right-of-way for improvements in cities with Federal funds in the name of city or town where land lies.	YES State may construct town and county highways including city streets, particular ways for which funds are appropriated, and highways in towns with valuation under $5 million, or if Federal funds are available, subject to consent of mayor or selectmen.	NO Except if counties or towns fail to do so, State may maintain and charge costs to local units.	State has same authority with regard to limited-access facilities as for State highways.	There is no legally designated State highway system. Department of Public Works lays out State highways in any location where it determines that public convenience and necessity require.
NO Except portion of highway fund allocated to cities and villages (through counties) for improvement and maintenance of roads and streets.	NO	NO	NO	State alone or in cooperation with cities and towns authorized to establish limited-access facilities, subject to municipal consent within cities and villages. State may provide for elimination of intersections with city and village streets by grade separation, access, or service road, or by closing off such streets at boundary line of limited-access highway but only with consent of city or village.	State Highway Commissioner may change, alter, or abandon location of any portion of any trunk highway, but must hear objections of county and city. State has been given authority from time to time to add additional mileage. Provision is made for cooperation with Federal Government in carrying out provisions of Federal-aid Act of 1916 and amendments thereto.
YES State-aid funds may be used for improvement of roads and streets in villages, boroughs, or cities if designated by county as State-aid roads.	NO	NO Except construction on streets over which trunk highways are routed, but which are not included in trunk highway system; must conform to State specifications as to grade and drainage.	NO	.....	State highway system set up in constitution, and legislature designates additional routes. State Highway Commissioner may locate routes added by legislature. Provision is made for cooperation with Federal Government in carrying out laws of United States and any rules and regulations made for expenditure of Federal funds.
NO	NO	NO	NO	.....	Legislature designates State highways but State Highway Commission may locate, relocate, widen, alter, change, etc.
NO Except when necessary to comply with any Federal law or requirement.	NO	NO Except when necessary to comply with any Federal law or requirement.	NO Except when necessary to comply with any Federal law or requirement.	The State Highway Commission has authority to limit access to, from, and across State highways.	State Highway Commission may make minor relocations of State highways provided they do not deviate from designated points established by legislature. Provision is made for cooperation with Federal Government to secure Federal funds.
NO But portion of motor-vehicle-registration fund is allocated to cities in specified population groups (through counties) for construction of permanent streets within incorporated limits.	NO	NO But may assist municipalities in construction of roads to and from municipal airports.	NO Except in connection with roads to municipal airports.	.....	State Highway Commission in conjunction with Board of County Commissioners designates State highways. In practice State highways through municipalities are constructed by the State Highway Department when Federal funds are involved but maintenance is by municipalities. A 1921 law provides for cooperation with Federal Government to obtain benefits of Federal-aid Act of 1916 and all other acts granting aid for public highways.
YES If Federal funds are involved. Part of motor-fuel-tax fund is allocated to incorporated places (through counties) for street improvement on basis of population. State also allocates funds for maintenance of city streets.	NO Unless Federal funds are involved.	NO Unless Federal funds are involved.	NO Unless Federal funds are involved.	.....	State highway system is designated by legislative determination. Apparently State highways are designated through all cities with population under 25,000.

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
NEVADA	YES By implication since State may finance improvements of State highways, which extend through municipalities.	YES By implication since State may acquire right-of-way for State highways, which extend through municipalities.	YES By implication since State is responsible for construction of State highways, which extend through municipalities.	YES By implication since State is responsible for maintenance of State highways, which extend through municipalities.
NEW HAMPSHIRE	NO Except in cities and towns under 2,500 population.	NO Damages for land taken for a highway are paid by town in which that part of highway is located.	NO Except in cities and towns under 2,500 population.	NO Except in cities and towns under 2,500 population.
NEW JERSEY	YES Or local bodies may assume portion of costs.	YES And may acquire land outside uniform width of highway to conform to comprehensive plan of municipalities.	YES Or may contract with municipalities to do work to conform to standards on State highway system. In cities over 75,000 population, city retains rights as to construction of utilities.	YES But in cities over 75,000 population city retains rights as to regulation of use of utilities.
NEW MEXICO	YES By implication since State may initiate and finance improvements on State highways, which are presumed to extend through cities.	YES By implication since State may acquire right-of-way for State highways, which are presumed to extend through cities.	YES By implication since State may initiate projects on State highway system considered neeessary, if county fails to do so, and State highways are presumed to extend through municipalities.	YES By implication since State may maintain State highways if sufficient funds are available, and State highways are presumed to extend through municipalities.
NEW YORK	YES With respect to certain arterial routes through cities, established by legislature, to width and type approved by State. State pays all construction costs and 50 percent of right-of-way costs.	YES With respect to certain arterial routes through cities (not including New York City).	YES With respect to certain arterial routes through cities, to width and type approved by State. Approval of city is necessary if land is to be acquired. Individual projects are authorized by legislature.	YES With respect to certain arterial routes through cities.
	State highways through towns and incorporated villages.	State may acquire land but changes of 1 mile or over are subject to approval of governing body of town or village.	To width approved by State. Additional width to be financed by local unit.	Maintenance and repair of improved State highways in towns and incorporated villages is under direct supervision of State.
	Certain cities of second and third class adjacent to New York City.	To be acquired by city or county.	To width of highway outside city.	To be maintained at expense of municipality.
NORTH CAROLINA	YES State apportions funds to cities and towns on basis of population, mileage of State highways, and need, to be used primarily for streets forming part of State highway system.	YES By implication since State has this authority for State highways, which extend through cities.	YES Or may contract with city or town to do work, but performance must meet State specifications.	YES Or may contract with city or town to do work, but performance must meet State specifications.

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
No	No	No	No		State Highway Department may supplement State highway system by establishing new routes into or in vicinity of municipalities and metropolitan areas with approval of county and city affected thereby. A 1945 amendment provides for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1944.
YES Part of motor-fuel-tax fund is allocated to towns for work on local roads and streets. State may finance reconstruction and maintenance of secondary State highways in cities and towns under 2,500 population. Aid is on sliding scale based on assessed valuation of cities or towns.	No	YES State has full control of reconstruction of secondary State highways in cities or towns under 2,500 population.	YES State maintains secondary State highways in cities and towns under 2,500 population or if Federal funds involved State may maintain in cities over 2,500 population if city fails to maintain properly.	State, with approval of governor and council, may designate and establish any existing street as included within a limited-access facility; may provide for elimination of intersections at grade with any existing street by grade separation or service road or by closing off such street; may plan, designate, establish, use, regulate, alter, improve, maintain, or discontinue any street, or designate as local service streets any existing street, and exercise jurisdiction over same in manner authorized over limited-access facilities.	Governor, with advice of council, determines whether there is occasion for laying out or altering State highways. A 1945 act provides for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1944.
YES State allocates portion of motor-vehicle fund to municipalities for construction of unimproved roads, on basis of population and mileage, and additional funds for special municipal projects.	No	NO Except that work done by municipalities with State funds allocated for construction of unimproved roads in cities and for special projects must conform to standards prescribed by State.	NO Except that municipalities accepting State allocations for construction of unimproved roads in cities must enter into agreement with State to keep roads in repair.	Freeways and parkways identified with State highways.	State highway system designated by legislature but commissioner of highways may add routes. Annual appropriation bill covers acceptance of any Federal funds available and authorizes State to carry out such projects as may be designated by the Federal Government.
No	No	No	No	State, alone or in cooperation with City of Santa Fe, may establish limited-access facilities and local service roads on U S Highways 85 and 64 and adjacent or connecting streets in and near Santa Fe subject to municipal consent within city limits. State may acquire right-of-way for highways or streets under its jurisdiction; may provide for elimination of intersections at grade between limited-access facilities and city streets and may determine what city streets may open into or connect with limited-access facility, subject to municipal consent in Santa Fe. State and city of Santa Fe may enter into agreement respecting financing, planning, establishment, improvement, maintenance, use, administration, or vacation of limited-access facility.	State highway engineer, under direction of State Highway Commission, selects and designates State highway system and may change from time to time subject to approval of State Highway Commission. According to law, "the balance of the moneys in the State road fund shall be used as the Commission shall determine for cooperating with the U. S. Government or with the several counties or municipalities of the State, or otherwise, in the discretion of the Commission for the construction, improvement, or maintenance of public highways and bridges."
YES State allocates part of motor-vehicle fund to villages and to New York City. 1/10 cent from motor-fuel tax allocated to New York City without restrictions as to use.	No	YES State may construct highways in villages to connect State highways with existing paved streets or highways, to width and type approved by State, upon petition of board of trustees of village. Upon request of municipality State may perform any work of construction or reconstruction in connection with performance of other improvements.	No	Thruways identified with State highways. State may establish and maintain arterial highways authorized by legislature through cities as part of State highway system. In New York City, State may construct arterial highways subject to city's approval of plans, etc., to be turned over to city after construction. Changes in basic plans involving abandoning or relocating sections, adding entrances or exits, or providing crossings at grade or traffic lights, other than on service roads, are subject to approval of State Superintendent of Public Works.	State highway routes designated by legislature.
YES Any balance of funds apportioned to cities and towns for improvement of State highways may be used for (first) important connecting links to State or county highway systems or farm-to-market roads and (second) for such streets as may be designated by municipality.	NO Not unless by implication for urban extensions of county highways, since State has this authority for county highways.	YES State may construct important connecting links to State or county highway system or farm-to-market roads, or streets designated by municipality, if funds are available, or may contract with municipality to do work, which must meet State specifications.	YES State may maintain important connecting links to State or county highway system or farm-to-market roads, or streets designated by municipality, if funds are available or may contract with municipality to do work, which must meet State specifications.		State Highway Commission may change, alter, add to, or discontinue roads included in State highway system. Provision is made for cooperation with Federal Government to secure benefits of Federal-aid act.

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
North Dakota	YES By implication since State is authorized to improve and maintain any part of State highway system, which extends through cities.	YES By implication since State may acquire land for State highways, which extend through cities.	YES To normal width of State highway. City may request additional width and pay difference.	YES To normal width of State highway. City may contract with State to maintain additional width.
Ohio	YES State allocates part of highway construction fund for urban extensions of State highways according to specified proportions.	YES By implication since State may acquire right-of-way for carrying out provisions of highway law.	YES Subject to approval of governing body of municipality, may construct, reconstruct, widen, and improve; may make improvements without municipal consent on State highways which are part of Federal-aid system if municipality refuses to cooperate.	YES Subject to approval of governing body of municipality.
Oklahoma	YES State has authority to construct and maintain extensions of State highways within incorporated limits of any city.	YES By implication since State may acquire land for State highways, which extend through cities.	YES State has authority to construct extensions of State highways within incorporated limits of any city.	YES State highways in cities to be maintained in same manner as other State highways.
Oregon	YES May construct and maintain streets through incorporated cities where such streets form links in State highway system or connection between two such highways and which streets have been designated by State as streets over which State highway traffic is routed.	YES May acquire right-of-way necessary for State highways within incorporated limits of city at sole expense of State.	YES May construct streets forming links in State highway system in incorporated cities but may not change grade without consent of governing body of city.	YES Has complete control over streets taken over from curb to curb or, if no established curb, over such portion of right-of-way as may be utilized for highway purposes.
Pennsylvania	YES With respect to streets in first-class cities taken over as State highways (city may pay part).	YES State pays for right-of-way, city acts as agent. (City may pay part of cost).	YES Shall construct, reconstruct, resurface, and repair.	YES But is under no obligation to maintain curb or footways, remove snow, or clean streets.
	All State highways within boroughs and incorporated towns.	County acquires right-of-way if agreeable to proposed improvement; otherwise State may do so.	Shall improve or reconstruct to width and type determined by State. May change, alter, or establish width, grades, or line.	Shall maintain to width and type determined by State.
	Designated streets in cities of second, second A, and third class which are part of State system (city may pay part).	County acquires right-of-way if agreeable; otherwise State may do so.	Shall construct, reconstruct, resurface to width between curbs. Changes in width, lines, or grades subject to approval of city.	To width between curbs. Maintenance does not include snow removal or street cleaning.
	Existing roads and streets in cities, boroughs, and townships approximately parallel to State highways which shall become part of State highway system.	County acquires right-of-way if agreeable; otherwise State may do so.	But in cities and boroughs approval of council is required for relocation, change in line or width, or for constructing wholly or in part on new alinement.	Maintained as State highways.
	Connecting roads (not over 1 mile in length) joining two or more State highways, in second, second A, and third-class cities, boroughs, incorporated towns, or townships, to lessen distance or improve alinement.	County acquires right-of-way if agreeable; otherwise State may do so.	May take over or build.	Maintained as State highways.
	May establish, construct, and maintain as State highways roads or streets parallel to existing State highways in townships, boroughs, incorporated towns, or cities.	County acquires right-of-way if agreeable; otherwise State may do so.	May establish and construct.	May maintain.
	May pay all or part of cost of improving streets and highways in cities of second, second A, and third class which are not State highways but continuations thereof (under agreement with city).	Presumably either State or city may acquire, depending on terms of agreement.	Under agreement with local unit, may construct or reconstruct to existing or lesser width; city may pay for greater width if desired.	May maintain under agreement with local unit.

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
YES State aid for counties (1 cent of motor-fuel tax) is to be used for "city streets leading up to and connecting with Federal-aid and State highways." Cities may also request State assistance in improving city streets by formation of improvement districts.	No	No Except that improvement districts may request State assistance in improving city streets.	No		State Highway Department has full power and authority to designate, locate, and create new and additional routes of State highway system. Provision is made for cooperation with Federal Government in order to obtain benefits of Federal-aid Highway Act of 1916 or any future acts.
YES State allocates funds from highway construction fund to municipalities for construction and maintenance of city streets, street cleaning, and traffic lights.	No	No	No	State Director of Highways has similar powers with respect to establishment and maintenance of limited-access highways and freeways as he possesses relative to other highways.	State Highway Department has authority to designate additional State highways or change existing State highways after public hearing. Provision is made for cooperation with Federal Government to secure benefits of Federal-aid Acts of 1916 and 1935 and acts amendatory thereto.
YES State allocates portion of motor-vehicle fund to counties for allocation to cities and incorporated towns, for construction, maintenance, repair, and improvement of streets and alleys. State also allocates 5 percent of motor-fuel-tax fund to incorporated cities and towns for construction and maintenance of streets and alleys (on basis of population).	No	No	No	Governing body of city over 5,000 population authorized to enter into agreement with State respecting financing, planning, establishment, improvement, maintenance, or vacation of limited-access facilities.	State highway system designated by State Highway Commission.
YES If Federal funds are available. State is authorized to construct, maintain, etc., streets forming links in secondary State highway system. State may enter into agreement with cities for construction and maintenance of streets on terms mutually agreed upon. State also allocates funds to cities for construction and maintenance of city streets which have not been designated as connecting links of State highway system (cities under 1,000 population must submit plans to State). Allocation is made on basis of population. $250,000 of this fund to be administered by State for improvement of streets not on State highway system which are receiving excessive wear.	YES State may acquire land for extensions of secondary State highways within corporate limits of cities and towns also for improvement districts.	YES If Federal funds are available. State may also construct streets on urban extensions of secondary State highway system, or streets receiving excessive wear through sudden increases in population or heavy-industry traffic.	YES If Federal funds are available. State may maintain urban extensions of State secondary highway system. State may enter into agreement with cities for maintenance of streets.	State's authority to construct throughways includes State highways within corporate limits of cities and towns and with approval of municipal authorities may extend to and include city streets. Access points to throughways to be designated by municipal authorities, but upon failure of such authorities to so do, within a given length of time, State may designate such points. Authority of State to provide service roads subject to municipal consent. State may enter into cooperative agreement with city for location, adoption, construction, and maintenance of throughway, both with respect to State highways and to streets under jurisdiction of incorporated cities and towns.	State Highway Commission authorized to select and designate streets over which State highways shall be routed, and may alter or change such routes when deemed advisable. "State highways" include secondary routes. Provision is made for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1916 and other acts "heretofore or hereafter enacted."
NO Except if Federal funds are involved. State also allocates portion of motor-license fund to cities of first class, second class, second class A, and third class, boroughs, and incorporated towns and townships of the first class, according to designated proportions. State also allocates part of liquid-fuels-tax fund to counties for county aid for city streets according to designated proportions.	No	NO Except if Federal funds are involved in which case State may relocate, widen, or construct as for State highways.	NO Except presumably if Federal funds are involved.	Secretary of Highways, with approval of governor, may declare any existing or hereafter established State highway to be a limited-access facility. Designation of such a highway in a city is subject to approval of city. Secretary of Highways may lay out and construct local service roads subject to approval of local authorities; may enter into agreements for sharing of cost of property damages with officials of any public subdivision of the Commonwealth. Political subdivisions of Commonwealth may make contribution to department of highways toward cost of establishment or improvement of limited-access highway or local service highway by the department of highways or toward cost of maintenance by department of highways.	Secretary of Highways, with approval of governor, may designate future location and width of any proposed State highway. Provision is made for cooperation with Federal Government to obtain benefits of Federal-aid Act of 1916 and its supplements and amendments, or any other act of Congress providing Federal aid for any highway purposes.

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
RHODE ISLAND	YES May construct, reconstruct, improve, and maintain as part of State highway system any street, road, avenue, or right-of-way in city which is extension or connection of existing State road into or through said city, subject to approval of city.	YES May condemn or purchase property for city extensions of State highways.	YES May construct, reconstruct, or improve extension or connection of existing State road into or through city, subject to approval of city.	YES State jurisdiction extends from curb to curb. State may relinquish maintenance to city.
SOUTH CAROLINA	YES But in cities over 2,500 population State may not spend over $500,000 per year, including funds for matching Federal aid for this purpose, nor any funds for this purpose until Federal aid for secondary and farm-to-market roads has been matched.	YES State may condemn land necessary for right-of-way for State highways in towns or municipalities.	YES But approval of local governing body is necessary in municipalities over 2,500 population. Construction includes necessary provision for operation and parking of vehicles, sidewalks, gutters, storm drains, etc.	YES Including sidewalks, gutters, storm drains, etc.
SOUTH DAKOTA	YES By implication, since highway fund is apportioned to counties for expenditure on State highways, which include urban extensions.	YES By implication since State acquires and pays for land necessary for widening, changing, relocating, constructing, reconstructing, maintaining, or reopening any portion of State highway system, which extends through municipalities.	YES By implication since highway fund is apportioned to counties for improvement of State highways (by State), which extend through municipalities.	NO Except in cities under 2,500 population.
TENNESSEE	YES State Highway Department to construct, maintain, or contribute to construction or maintenance of all streets or roads in municipalities over which traffic from State highways is routed.	YES State may enter into contract with municipalities relative to acquisition of right-of-way for streets in municipalities over which traffic from State highways is routed.	YES To width and type State may deem proper, but such width shall not be less than 18 feet.	YES But city may maintain if equipped to do so and State will reimburse.
TEXAS	YES May enter into agreement with governing bodies of incorporated cities, towns, and villages providing for location, relocation, construction, control, supervision, and regulation of designated State highways within or through corporate limits of municipalities, and determine respective liabilities.	YES By implication since State may acquire land for State highways, which extend through municipalities.	YES Under agreement with municipalities.	YES Under agreement with municipalities.
UTAH	YES All roads or streets designated as State roads by legislative action, to be constructed and maintained by State with funds made available for that purpose. State to make agreements with any city or town for rights-of-way, for improvement or maintenance of State roads at expense of State, city, or town, or joint expense.	YES State to make agreements with any city or town for rights-of-way for State roads at expense of State, city, or town, or joint expense.	YES All roads or streets designated as State roads by legislative action to be constructed by State. State to make agreements with any city or town for construction of State roads. Where public highways extend through incorporated cities or towns they must conform to direction and grade and be subject to regulations of other streets in such cities or towns.	YES All roads or streets designated as State roads by legislative action to be maintained by State. State to make agreements with any city or town for maintenance of State roads.
VERMONT	NO Except where population is under 1,500 or houses are 100 feet or more apart. State also makes annual appropriation to towns for improvement of State-aid highways.	NO Unless by implication, where population is under 1,500 or houses are 100 feet or more apart, since State has this authority for State highways, which extend through towns in this category.	NO Except where population is under 1,500 or houses are 100 feet or more apart; also State may supervise expenditure of State-aid funds for "State-aid connecting links."	NO Except where population is under 1,500 or houses are 100 feet or more apart; also State may supervise maintenance of "State-aid connecting links" with State-aid funds.

in municipalities, and related provisions, 1948—**Continued**

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
No	No	No	No	State's authority regarding limited-access facilities same as for other highways of State. Access to freeway from existing street may be regulated and restricted by Director of Public Works and Division of Roads and Bridges. Access to freeway from new street is subject to consent and approval of said director.	State highway system and any additions designated by legislature. Provision is made for cooperation with Federal Government to obtain benefits of Federal-aid apportionments.
No Except State may use State funds for belt lines or spurs of 2 miles or less; also for streets or roads at State institutions.	No	No Except State may construct belt lines and spurs of 2 miles or less; may also hard-surface streets or roads including sidewalks at State institutions.	No Except State may maintain belt lines and spurs of 2 miles or less constructed by State.	----------	State highway primary system to consist of connected system of principal State highways not to exceed 10,000 miles, connecting centers of population, as determined by State Highway Commisssion. State highway secondary system to consist of all State highways not included in State primary system. State Highway Commission may transfer route from secondary to primary system if deemed desirable to better serve the traveling public.
No	No	No	No	----------	State Highway Commission may, at its discretion, extend State highway system to include any street or streets within limits of any city if necessary to make continuous route, but may not generally add routes to system. Provision is made for cooperation with Federal Government to obtain benefits of Federal-aid Highway Act of 1916.
No	No	No	No	----------	State Highway Commissioner authorized to add routes to State highway system. Provision is made for cooperation with Federal Government to receive Federal funds.
YES But only if Federal funds are available, and subject to municipal consent.	No	YES But only if Federal funds are available, and subject to municipal consent.	YES But only if Federal funds are available, and subject to municipal consent.	Authority to establish freeways rests with cities and towns over 175,000 population.	State Highway Commission may add routes to State highway system. In practice State responsibility for State highways in cities extends from curb to curb. State is not responsible for congested or downtown sections in cities over 15,000 population. Cities are responsible for acquisition of land for improvements on State highway system.
No But State appropriates money to cities for "Class C" roads and furnishes standard plans and specifications.	No	No But cities of third class may contract with State for construction of "Class C" roads within corporate limits. State to cooperate with governing officials in construction of "Class C" roads.	No But cities of third class may contract with State for maintenance of "Class C" roads within corporate limits. State to cooperate with governing officials in maintenance of "Class C" roads.	The State, acting alone or in cooperation with local agencies, is authorized to participate in establishment of limited-access facilities and local service roads, provided that within cities and towns, such authority shall be subject to such municipal consent as may be provided by law. The State and cities and towns are authorized to enter into agreement with each other regarding the financing, planning, establishment, improvement, maintenance, use, regulation, or vacation of limited-access facilities.	State highway routes are designated by legislature. "Class A" roads are State highways. "Class B" roads are county highways. "Class C" roads are all others. State Road Commission authorized to enter into all contracts with Federal Government to comply with Federal ac's.
YES State appropriates funds for expenditure on town highways on mileage basis.	No	YES Construction with State funds is under supervision of State highway department.	No	----------	Legislature designates State highways but State Highway Board may add routes under certain conditions. Extensions of State highways through municipalities are State-aid highways or "State-aid connecting links."

State	State authority for urban extensions of State primary highways			
	Financing	Acquisition of right-of-way in name of State	Construction	Maintenance
VIRGINIA	YES State may pay 50 percent of cost of construction and improvement, including right-of-way for bypasses, extensions, or connections of State highways in cities over 3,500 population, and 100 percent for extensions of State highways in incorporated towns and cities under 3,500 population. State also allocates additional funds to cities and towns over 3,500 population for maintenance and improvement including construction and reconstruction of streets, roads, and bridges, on mileage basis.	YES In connection with bypasses, extensions, or connections of primary highway system through or around cities or incorporated towns.	YES In connection with bypasses, extensions, or connections of State highway system through or around cities and incorporated towns. Plans and specifications for construction with additional funds allocated on mileage basis are subject to approval of State in cities and towns.	YES In connection with all State highways in incorporated cities and towns under 3,500 population. Cities over 3,500 population maintain bypasses, connections, or extensions of State highway system but State contributes such sums as may be provided by law.
WASHINGTON	YES May spend any State funds available on city streets in incorporated cities and towns forming part of State highway system. State allocates portion of motor-vehicle fund to cities and towns on basis of population for improvement of primary and secondary State highways.	YES On city streets in incorporated cities and towns forming part of State highway system (State "shall" acquire right-of-way for improvement of urban extensions of primary State highways in incorporated cities and towns under 1,500 population).	YES On city streets in incorporated cities and towns forming part of State highway system (State "shall" construct extensions of State primary highways in incorporated cities and towns under 1,500 population).	YES On city streets in incorporated cities and towns forming part of State highway system (State "shall" maintain extensions of State highways in incorporated cities and towns under 1,500 population). City may authorize State to maintain city streets with funds from State appropriation if city is unable to do so.
WEST VIRGINIA	YES But assumes no greater obligation for connecting parts of State road system than it is required to assume outside municipalities.	YES By implication since State has this authority for all State roads, which extend through municipalities.	YES But shall assume no greater obligation for connecting parts of State road system than it is required to assume outside of municipalities.	YES But shall assume no greater obligation for connecting parts of State road system than it is required to assume outside of municipalities.
WISCONSIN	YES On State highway extensions in cities under 2,500 population and on connecting streets in cities over 2,500 population (subject to approval of counties if county allotment is used). Cities and villages receive allocation on mileage basis for maintenance and construction of streets connecting but not part of State highway system.	NO. Necessary land to be acquired by board, commission, or department selected by city subject to approval of State. State is grantee.	YES On State highways in cities under 2,500 population and on connecting streets in cities over 2,500 population, subject to approval of counties if funds from county allotment are used. State shall construct in same manner as portions of State trunk highway system (compulsory only to same width).	NO Except on State highways in cities under 2,500 population.
WYOMING	YES On streets constituting direct connections of State highways in cities under 1,500 population, State pays entire cost of construction and maintenance. In cities over 1,500 population, city pays half of construction cost and all of maintenance cost.	YES By implication since State has this authority for State highways.	YES All work to be performed under direct supervision and control of State. State may cooperate with incorporated cities, towns, and villages in construction of public streets and highways which constitute direct connections between sections of established State highways.	NO Except in cities under 1,500 population.

in municipalities, and related provisions, 1948—Continued

State authority for urban highways other than extensions of State primary system				State authority for urban controlled-access highways	Supplementary comments
Financing	Acquisition of right-of-way in name of State	Construction	Maintenance		
YES State may finance connecting links of secondary system in incorporated towns under 3,500 population.	YES State may acquire land for connecting links of secondary system in incorporated cities and towns under 3,500 population.	YES State may construct connecting links of secondary system in incorporated cities and towns under 3,500 population.	YES State may maintain connecting links of secondary system in incorporated cities and towns under 3,500 population.	State's authority regarding limited-access highways same as for other highways.	Designation of State highways is by legislative enactment, but State may transfer total of 50 miles of State secondary roads to primary system per year. State Highway Commission authorized to comply with provisions of present or future Federal-aid acts.
YES State may expend any funds available on secondary State highways in incorporated cities and towns. State also apportions part of motor-vehicle fund for improvement of city streets (on basis of population).	YES State may acquire land for secondary State highways in incorporated cities and towns (State "shall" acquire land for improvement of urban extensions of secondary State highways if population is under 1,500).	YES State may construct secondary State highways in incorporated cities and towns (State "shall" construct urban extensions of State secondary highways if population is under 1,500). City may authorize State to construct city streets with funds from State appropriations if city is unable to do so.	YES State may maintain secondary State highways in incorporated cities and towns (State "shall" maintain urban extensions of State secondary highways if population is under 1,500). City may authorize State to maintain city streets with funds from State appropriation if city is not equipped to do so.	State's authority regarding limited-access highway facilities same as for other highways. Within incorporated cities State authority is subject to consent of local authorities.	State highway routes are designated by legislature but State Division of Highways designates routes through incorporated cities and towns. Secondary State highways are defined as branches of primary State highways.
YES State may finance connecting parts of secondary State highways to extent required outside municipalities.	YES State may acquire land for connecting parts of secondary State highways to extent required outside municipalities.	YES State may construct connecting parts of secondary State highways to extent required outside municipalities.	YES State may maintain connecting parts of secondary State highways to extent required outside municipalities.	State's authority regarding freeways is same as that for other highways. Access to any freeway from any new street shall be established by and with consent of State Road Commission.	State highway system designated by State Road Commission. State has control over all primary and secondary State highways (Secondary State system is former county district system). Provision is made for cooperation with Federal Government to secure benefits of Federal-aid Act of 1921 and all subsequent acts.
YES If Federal funds are available. State also allocates portion of State highway fund to towns, villages, and cities, on mileage and population basis, for improvement of roads and streets not on State or county systems. County appropriation may be used by Milwaukee County for improvement of city streets.	NO	YES If Federal funds are available.	NO Maintenance of other than State highways is responsibility of counties, cities, etc.	----------	State Highway Commission may revise State highway routes designated by legislature but revisions over one-half mile must be approved by county. State trunk highways extend through cities under 2,500 population. (Called connecting streets in larger cities). Provision is made for cooperation with Federal Government to secure benefits of Federal-aid acts.
YES If Federal funds involved. State allocates part of motor-fuel-tax fund to cities and towns for construction and maintenance of streets if population is over 1,500. Apportioned on basis of population.	NO	YES If Federal funds involved.	NO	----------	State Highway Commission authorized to designate State highways. Provision is made for cooperation with Federal Government to secure benefits of Federal-aid Act of 1916 and all acts amendatory and supplementary thereto.

Continued from page 165)

In only five States [22] is the State highway system designated by the legislature with no provision under which the State highway department may add additional routes or alter those designated by the legislature, which apparently reserves this authority to itself. In the remaining States (with the exception of New Hampshire [23]) the State highway department either has authority to designate the system itself or may add individual routes or relocate those previously designated.

In 13 States, whether the original system was designated by the legislature or by the State highway department, some authority is given to the department relative to selection or revision of urban extensions of the State primary system.[24] For example, the Arizona State Highway Department and the governing bodies of cities may select routes of State highways through cities. Likewise, the Georgia State Highway Department can designate State highways, including streets, "with proper notice to the governing body." The Indiana State Highway Commission has the authority to select routes of State highways through incorporated cities and may alter such routes for the convenience of traffic.

Comparable authority also exists in Louisiana, for the Department of Highways in that State may add routes to the State highway system and may include streets in cities forming continuations of or connections with State highways. The Nevada Department of Highways may establish new routes into or in the vicinity of municipalities and metropolitan areas subject to the approval of the local units involved.

The Director of Highways in Washington designates routes through incorporated cities, while the Illinois Department of Public Works and Buildings may relocate urban extensions to better serve traffic demands.

CONTROLLED-ACCESS HIGHWAYS

The last decade has witnessed the development of the controlled-access highway, particularly in the urban areas. Because many of these modern road facilities are being established by the State, it is important to know what legislative authorization exists with respect to such State expressways in municipalities.

Controlled-access highways are now sanctioned by legislative act in 26 States,[25] by constitutional provision in one State, Missouri, and by judicial decision in an additional State, Minnesota.

In 18 of these States,[26] the statute specifically identifies controlled-access highways with State highways. Accordingly, it is presumed that generally speaking, the same authority, or qualifications on authority, of State highway departments in urban areas apply to controlled-access highways as concern all State highways, unless the contrary is specifically stated. This may also be true of the remaining States that have sanctioned the controlled-access highway, but that fact is not specifically spelled out in their statutes.

Legislation frequently authorizes agreements between the State highway department and municipalities concerning the establishment of urban expressways and the consent of the city is often required. The laws of 16 States [27] now contain varying provisions of this character.

The Florida statute is typical. It provides that State highway authorities may establish controlled-access facilities and local service roads wherever they are of the opinion that traffic conditions warrant, subject to municipal consent within incorporated cities and towns; and are authorized to enter into agreement with cities, towns, or villages respecting the financing, planning, establishment, improvement, maintenance, use, regulation, or vacation of such facilities.

A variation of this provision is found in a few States, of which Colorado is illustrative. Under the Colorado law, the State highway engineer, with approval of the Governor, is authorized to enter into agreements with cities or towns having jurisdiction over city or town streets, concerning the closing of any city street at or near the point of intersection with the freeway; or to make provision for carrying such city street over or under or to a connection with the freeway; and to do any and all work on such city street as is necessary therefor.

Comparable provisions in other jurisdictions may be found in table 3, pages 166–81.

A rather unique provision relating to expressways through urban areas is to be found in the State of New York. In connection with the arterial highway program, the State is authorized by law to establish and maintain arterial highways through cities as part of the State highway system. Certain qualifications on the State's authority in New York City are found in table 3.

In some States, legislative authority with respect to controlled-access highways is limited to designated cities or urban areas of a stipulated population. In New Mexico, for example, only the city of Santa Fe is authorized to participate in the expressway program with the State, and then only on two specified routes and adjacent or connecting streets. In Oklahoma, cities over 5,000 population are authorized to act, more or less independently, though cooperation with the State Department of Highways is permitted. Authority in Texas rests only with cities and towns over 175,000 population.

OTHER URBAN HIGHWAYS

The legislative authority of the State highway departments is decidedly limited with respect to urban highways other than extensions of the State primary system.

Federal-aid Highways

Sometimes, State law specifically authorizes the State highway department to cooperate with municipalities in the expenditure of Federal funds. In California, for example, the Division of Highways may execute agreements with any city providing for the acquisition of property for, construction, improvement, and maintenance of any highway, including those not in the State highway system, to be constructed with Federal-aid funds. More general legislative language in Georgia permits the State Highway Department to cooperate with the Federal Government in planning, supervising, locating, improving, and constructing roads and streets in any part, section, or area of the State. Similar provisions are found in the statutes of 11 other States.[28]

Connecting or Feeder Streets

Somewhat restrictive are the laws in five States, involving the improvement of streets connecting or providing access to urban extensions of the State system. Illustrative of this type of legislation is the enactment in Kentucky authorizing the Commissioner of Highways to designate such city streets or portions thereof as in his discretion are feeder streets necessary to produce an adequate system of highways, and the State Department of Highways is empowered to construct and reconstruct, repair, and maintain such city streets, and make appropriations out of its funds for such purposes.

A similar law in South Carolina permits the State Highway Department to establish such belt lines or spurs of less than 2 miles in length as it deems proper, and to construct and maintain such facilities. The Department of Public Works in New York may construct roads or streets in villages, to connect State highways with existing paved streets. A Louisiana statute authorizes the Department of Highways to construct spur or stub connections from State highways to the official business district, where none exist. North Carolina may construct and maintain important connecting links to State or county highway systems.

Roads to State Institutions

Four jurisdictions are permitted to establish highway facilities in connection with State institutions. Alabama law, for instance, authorizes the State to improve and maintain streets abutting State educational or charitable institutions. Even broader is a Dela-

[22] California, Nebraska, New York, Rhode Island, and Utah.

[23] The Governor of New Hampshire, with advice of council, determines whether there is occasion for laying out or altering State highways.

[24] Arizona, Florida, Georgia, Idaho, Illinois, Indiana, Kansas, Kentucky, Louisiana, Nevada, Oregon, South Dakota, and Washington.

[25] California, Colorado, Connecticut, Florida, Illinois, Indiana, Kentucky, Louisiana, Maine, Maryland, Massachusetts, Michigan, New Hampshire, New Jersey, New Mexico, New York, Ohio, Oklahoma, Oregon, Pennsylvania, Rhode Island, Texas, Utah, Virginia, Washington, and West Virginia.

[26] California, Colorado, Connecticut, Florida, Illinois, Louisiana, Maine, Maryland, Massachusetts, New Jersey, New York, Ohio, Oregon, Pennsylvania, Rhode Island, Virginia, Washington, and West Virginia.

[27] California, Colorado, Florida, Illinois, Indiana, Kentucky, Louisiana, Michigan, New Mexico, New York, Oklahoma, Oregon, Pennsylvania, Texas, Utah, and Washington.

[28] Idaho, Indiana, Iowa, Kansas, Massachusetts, Missouri Nebraska, Oregon, Pennsylvania, Texas, and Wisconsin.

ware enactment involving roads leading from any public road to any institution owned or controlled in whole or in part by the State. Under Montana law, the State Highway Commission may assist municipalities in the construction of roads to and from municipal airports.

Some State authority is restricted to designated improvements, e. g., a road to serve the State College of Washington.

Secondary or State-aid Roads

The State highway departments in seven States [29] have some legislative authority to finance, acquire lands for, construct, and maintain urban extensions of secondary State highways or State-aid roads. Virginia, however, is empowered to act only in incorporated cities and towns under 3,500 population, while in West Virginia, authority with respect to the extent of improvement within municipalities is limited to that required outside such municipalities. Statutory provisions for all of these States are included in table 3.

ALLOCATION OF HIGHWAY FUNDS TO URBAN AREAS

More than half of the States allocate some portion of the revenues derived from highway-user imposts to urban areas, often with express provision for the improvement of urban highways other than extensions of State highways.

Broad authority is granted by law to the North Carolina State Highway and Public Works Commission, permitting the balance of State funds allocated to cities, after satisfactory improvement of local extensions of the State highway system, to be used for the betterment of streets which form important connecting links to the State highway system, the county highway system, or farm-to-market roads The State or the city or town may undertake the improvement of such facilities, but in any event construction must be in accordance with State specifications. Any remainder shall be used for the maintenance, repair, improvement, construction, or widening of any streets designated by the governing body of the municipality.

Somewhat of the same tenor is the California law under which five-eighths of 1 cent of the gasoline tax is allocated for expenditure within the various cities of the State, on a population basis, to be spent on major city streets that are not State highways, and on the secondary street system. The law provides that the expenditure of these funds shall be delegated by the State to the governing body of the city, if the State Division of Highways is satisfied that the city is equipped to conduct the work in an efficient manner. Otherwise, the work is to be performed by the State itself.

Three-fifths of the money apportioned to cities in California is restricted to construction expenditures on major city street systems, while the remaining two-fifths may be used for maintenance expenditures on either the major or secondary city street systems. No money apportioned to cities is earmarked for State highways, because the State is charged with the responsibility of constructing and maintaining such urban facilities.

In Illinois, approximately one-third of the motor-fuel-tax fund is allocated to the municipalities of the State for the construction and maintenance of a system of arterial streets or thoroughfares, other than urban extensions of State highways, subject to the approval of the Department of Public Works and Buildings. Improvements made on unimproved roads by cities in New Jersey, with funds allocated from the motor-vehicle fund, must conform to standards set by the State.

A unique provision has recently been added to the Oregon code, to the effect that the State may construct streets, other than State highways, receiving excessive wear through sudden increases in population or heavy industrial traffic, from a special fund designated for the purpose.

In general, however, few conditions attach to State allocations of highway-user imposts to the urban areas. Funds are apportioned most often "to cities and towns," "to cities for highway purposes," "to cities for improvement and maintenance of city streets," or according to similar provisions.

It might be mentioned, parenthetically, that a trend seems discernible toward the allocation of increased revenues from highway-user imposts to cities, though not necessarily on a percentage basis. Six States enacted legislation in 1947 increasing their highway revenue allocations to the urban areas.[30]

A comprehensive analysis of the authority of the States, as used administratively, is found in the Public Roads Administration's *Highway Statistics, 1947*,[31] in the section on highway taxation, particularly tables G–106, MV–106, and MC–106 (pp. 28–46).

URBAN MILEAGE UNDER STATE CONTROL

In order to provide appropriate background data for this analysis of statutory authority of State highway departments in urban areas, the compilation shown in table 2 (page 165) was made of the urban mileage under State control and its relationship to the State primary systems and to the total urban mileage in each State. The table is based on Public Roads Administration data collected from the States.

Mileage shown for State primary systems represents the number of miles for which the State actually assumes administrative control, and does not necessarily represent the entire mileage over which the State has statutory authority.

SUMMARY OF STATE STATUTORY AUTHORITY

Table 3 (pp. 166–81) reports in summary, for each State, the statutory authority of the State highway departments in municipalities, and related provisions, as of January 1, 1948. The information reported is based in part on an analysis of pertinent sections of the respective State statutory codes, and in part on a circularization of the State highway departments. Some of the provisions include administrative practice pursuant to law.

[29] Connecticut, Maine, New Hampshire, Oregon, Virginia, Washington, and West Virginia.

[30] California, Colorado, Indiana, Maryland, New Jersey, and Oregon.

[31] *Highway Statistics, 1947*, compiled by the Public Roads Administration; Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., 45 cents.

HIGHWAY PRACTICE
in the
United States of America

A 230-page, well-illustrated bulletin, *Highway Practice in the United States of America*, has been prepared by the staff of the Public Roads Administration for the use of foreign engineers who come to the United States to study and observe highway practice as it has developed in this country, and for all students, engineers, and officials interested in highways. It includes extensive discussions of highway history, administration, and finance; systems and standards; location and design; and construction and maintenance. Bibliographies are appended.

Copies of the bulletin may be purchased from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 45 cents per copy. Prepayment is required. Do not send orders to the Public Roads Administration. No general free distribution will be made.

U. S. GOVERNMENT PRINTING OFFICE: 1949

A Cooperative Study of Structural Design of Nonrigid Pavements

Second Progress Report

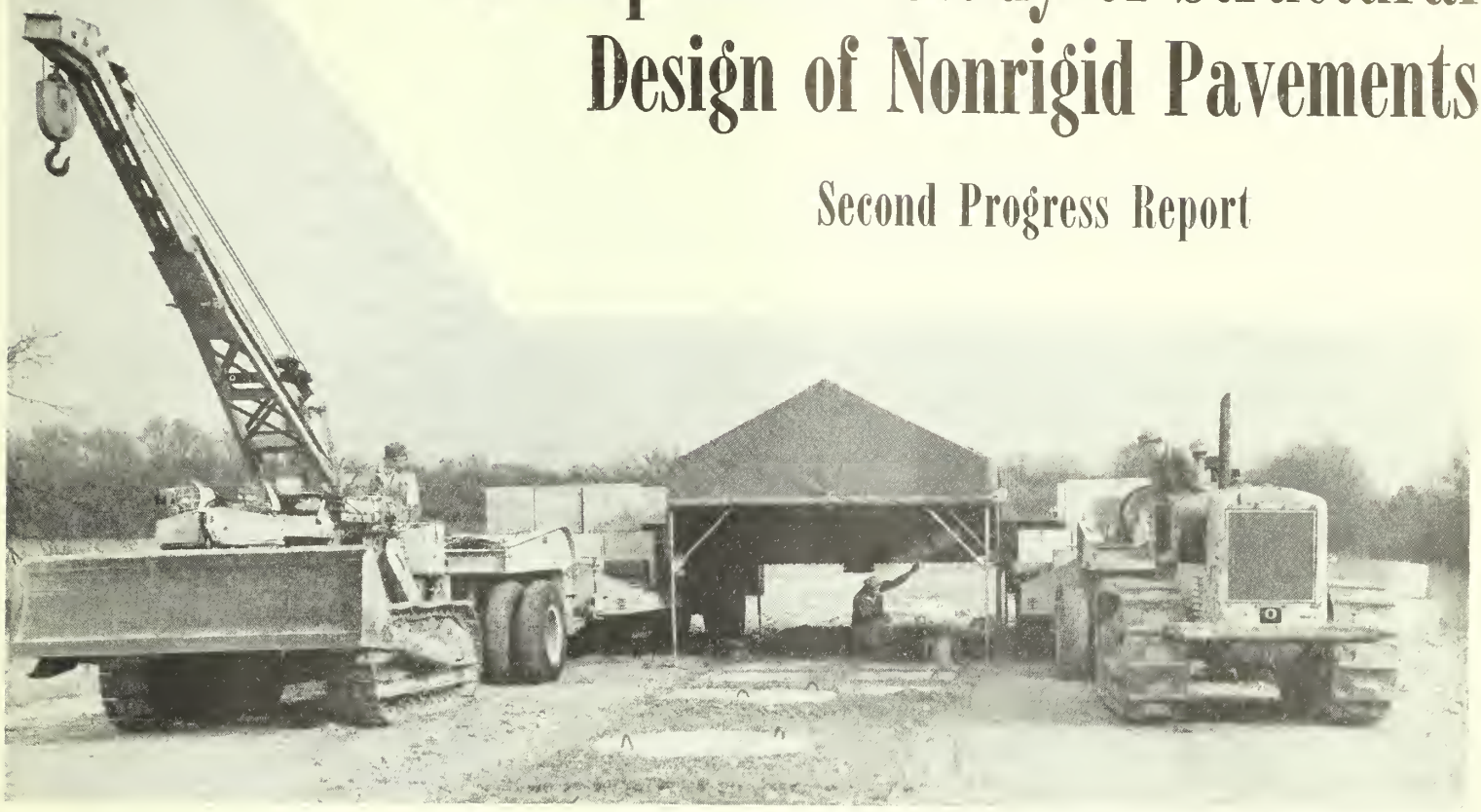

Figure 1.—The load reaction trailers

A DESCRIPTION OF FURTHER DEVELOPMENTS IN TEST METHODS AND INSTRUMENTATION

Reported by R. J. LANCASTER, Highway Research Engineer, Bureau of Public Roads and J. E. DRISCOLL, Research Engineer, the Asphalt Institute

This article is a second progress report on the cooperative investigation of the structural design of nonrigid pavements. The extensive experiments will result in the collection of a large body of important data and, it is hoped, in the development of significant conclusions of great value.

The principal objectives of the investigation include development of the load-supporting values of nonrigid pavements by full-scale field tests and correlation of these data with laboratory tests and with in-place determinations of various values of the base-course and subgrade components.

The first progress report provided a description of the investigation, its purposes, and proposed methods of procedure. This second progress report describes some innovations in plate-load testing techniques and equipment, including a relatively rapid method of testing which gives both complete load-deflection relations and information pertaining to the behavior of the components of a nonrigid pavement.

THE introductory report of this study was presented in PUBLIC ROADS over a year ago.[1] In addition to outlining the objectives of the study it included a descriptive account of the construction of the test pavement and a discussion dealing with the development of test apparatus and testing techniques.

The accumulation of test data from the investigation has not progressed to a point that warrants presentation of the findings at this time.[2] However, it is felt that a description of further developments in test methods and instrumentation might currently prove of interest and value to others carrying on research work in the same field. This report, then, consists principally of a detailed description of the methods of load testing that have been used in the study to date.

The study is a cooperative undertaking of the Highway Research Board, the Asphalt Institute, and the Bureau of Public Roads It is being conducted on an oval test track having two parallel tangents 800 feet in length connected at each end by a circular curve of 200-foot radius. The site of the experiment is near Hybla Valley, Va., 10 miles south of Washington, D. C.

The salient features of the pavement sections being tested at the present time are shown in figure 2. They consist of four main sections, each 200 feet in length and 44 feet in width, having base-course and surface-course thicknesses as indicated. The sections of pavement were built upon an embankment of uniform A–6 soil. The stabilized-aggregate base mixture was designed to conform with specification M 56–42, Type B–1, of the American Association of State Highway Officials, modified to permit the use of a slight amount of oversize material. The bituminous concrete was of the hot plant-mixed type, conforming to the Asphalt Institute dense graded aggregate specification A–2–a. Mix No. IV of this specification was used for all 3-inch surfacing and for the top 3 inches of

[1] *A cooperative study of structural design of nonrigid pavements*, by A. C. Benkelman and F. R. Olmstead, PUBLIC ROADS, vol. 25, No. 2, December 1947.

[2] This paper was presented at the 28th annual meeting of the Highway Research Board, December 1948.

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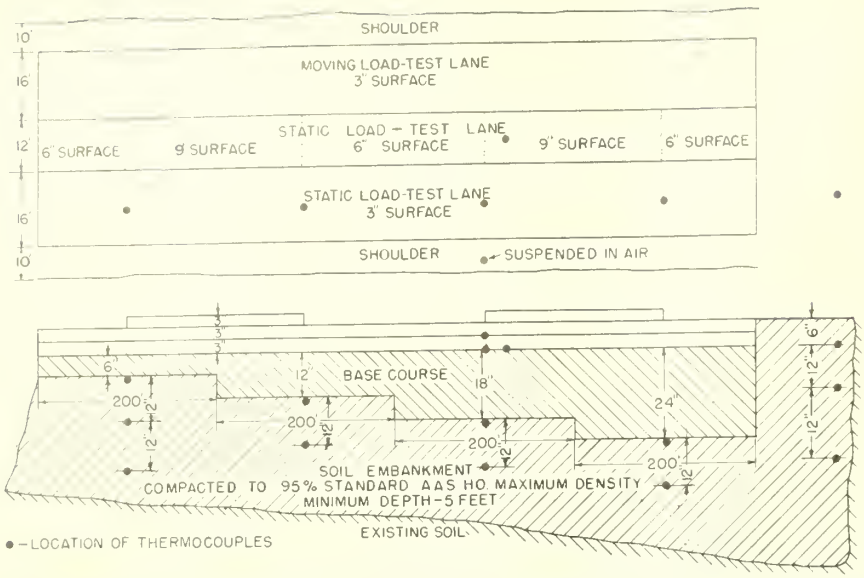


Figure 2.—Plan and profile of pavement sections, and locations of thermocouples.

the 6- and 9-inch surfacing. Mix No. II was used for the remaining lower portions of the 6- and 9-inch surfacing.

ORIGINAL TESTING PLANS

The original plans of the investigation called for the conduct of tests of both the static and moving-load type. The static or plate tests, totaling about 500, were to be made first. It was realized that the execution of these tests would require a very long period of time, particularly if but one test could be made per working day and if, because of climatic conditions, testing had to be restricted to a certain season of the year. A series of exploratory tests had demonstrated clearly that consistent results could not be anticipated if tests on the asphaltic concrete were conducted at pavement temperatures appreciably below 80° or above 90° F. In the region of Washington, D. C., pavement temperatures of this range in an area shaded from the sun prevail generally during the four summer months only, or for a period of about 80 working days.

In the early stages of the investigation consideration was given to the development and possible use of some type of load-bearing surface that would more closely simulate the effect of loads applied through tires than could be obtained with rigid plates. Several types of bearing surfaces were developed that gave promise of accomplishing the desired result. However, the use of another innovation of testing (simultaneous deflection measurements) which will be described later rendered their use impractical and for this reason all of the regularly scheduled bearing tests to date have been made with rigid plates. Four sizes of plates were selected, having diameters of 12, 18, 24, and 30 inches respectively. Two 25-ton capacity low-bed trailers were procured which, when loaded with concrete blocks and cross-connected with a heavy reaction girder (see fig. 1) provided a fairly mobile reaction load up to the 110,000-pound maximum capacity of the available hydraulic jacking system.

INCREMENTAL-REPETITIONAL LOADING TESTS

During the summer of 1947 about 60 plate-load tests were made, using an incremental-repetitional procedure. This procedure, which was mentioned in the first progress report, provided for the application of four load increments, each increment being applied and released five times. The magnitude of each load increment is such as to produce an increment of deflection of 0.125 inch. Thus the gross deflection of the material being tested will total 0.5 inch. As a part of this procedure, each individual period of load application and of load release is maintained until the rate of vertical movement conforms to a specified criterion. Obviously the time necessary to complete a test depends to a great extent on the specific rate value selected for this criterion. For the 1947 tests, this value was 0.001 inch per minute and for the higher load increments a time interval of as much as one-half hour was required to meet the specified criterion. The total time required for making a test using this procedure varied somewhat, depending upon the size of plate and character of the material being loaded. However, it was found that not more than one test on an average could be completed per working day.

Typical data obtained with this method of test are shown in figure 3. This is a load-deformation plot of the surface of the pavement only. Comparable data were obtained simultaneously at the surfaces of the base course and of the subgrade, so that two additional graphs of this nature are required to show all the data resulting from one test.

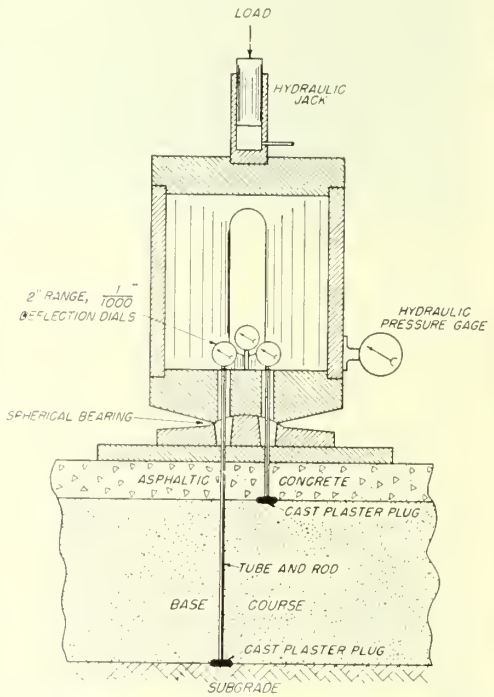


Figure 4.—Method of measuring simultaneous deflections at three levels of the pavement structure.

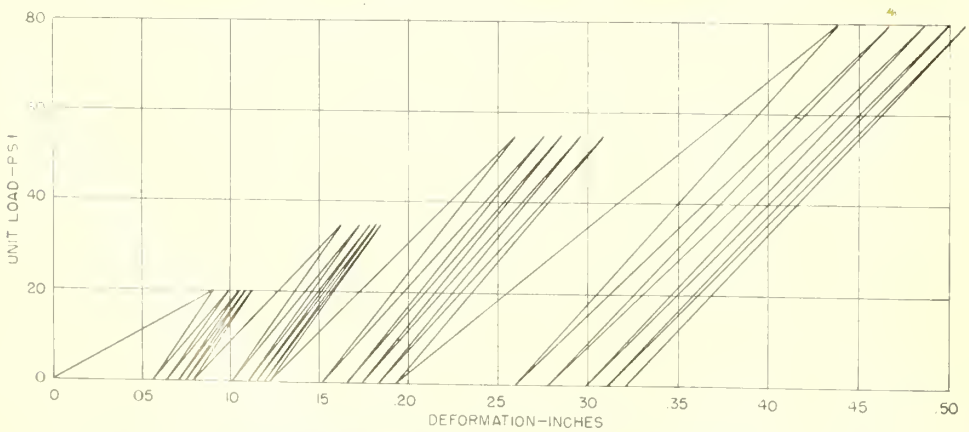


Figure 3.—Typical load-deflection and load-settlement data: 1947 procedure.

ACCELERATED LOAD-TEST PROCEDURE

Mention was made in the first progress report of a rapid method of testing that was the subject of some experimentation during preliminary stages of the investigation.

The outstanding feature of this so-called accelerated method of test involved the continuous application of load until the resistance of the material was overcome. The rate of application of the load was controlled so as to produce a constant rate of deflection of the material under test of 1.2 inches per minute. This meant that with an ultimate or end-point deflection of 2 inches, the actual time involved

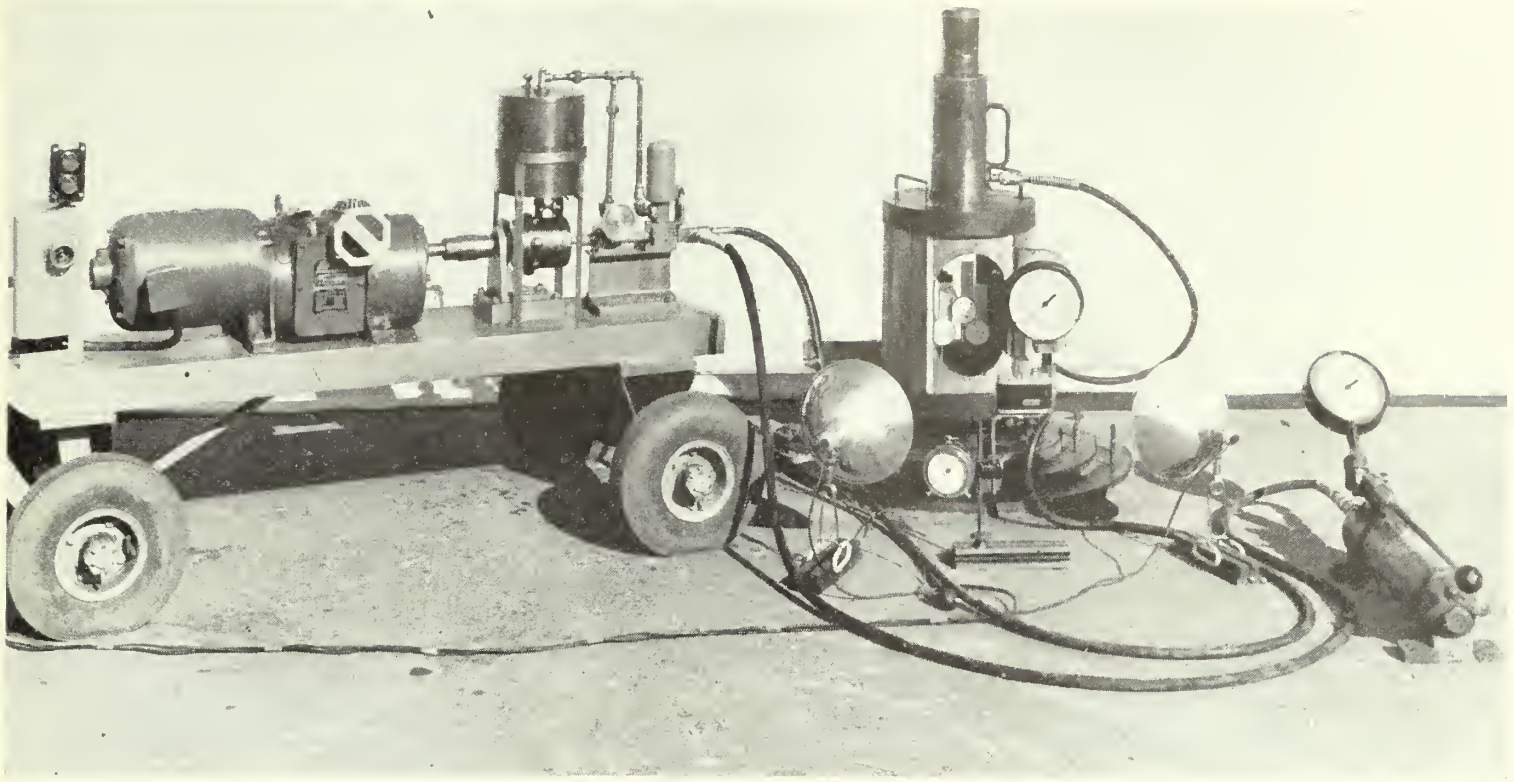


Figure 5.—Typical load-test set-up.

in making a test was less than 2 minutes, and that with adequate personnel and equipment a very large number of tests could be made in one day.

The results of the preliminary work using the accelerated procedure of loading were unusually well defined with respect to the effect of the size of loaded area and thickness of the pavement, the two most important variables being studied. Also, as a general rule, excellent agreement was found to exist between the results of duplicate tests. Moreover, with this testing procedure ultimate load values are obtained—information which it is believed will prove helpful in the analysis and correlation of the pavement load-test data.

The points enumerated above all entered into the decision to conduct a comprehensive series of tests utilizing the principal features of the accelerated test. However, the procedure as outlined above was modified to include the application and release of three load increments prior to applying the load continuously. This modification was introdueed in order to obtain some information on the elastie aetion of the pavement components at loads less than ultimate.

STEPS IN PROCEDURE

The procedure, as modified and adopted for the 1948 series of tests, consists of the following steps:

1. Application of load sufficient to produce a deflection of approximately 0.2 inch.
2. Maintenance of the load until the rate of deflection slows to 0.001 inch in 15 seconds.
3. Release of the load until the rebound slows to 0.001 inch in 15 seconds.
4. Repeat steps 1 to 3, with deflection of of 0.3 inch in step 1.
5. Repeat steps 1 to 3, with deflection of 0.4 inch in step 1.
6. Continuous application of load so as to produce a rate of deflection of 0.5 inch per minute until a deflection of 2 inches is reached.
7. At a location just outside the influence of the test load the moisture content and density of the base course are determined, an undisturbed core of the subgrade is taken for a triaxial test, and moisture samples of the subgrade are taken from depths ranging up to 3 feet.

The word "deflection" as used above refers to the downward movement of the bearing plate, under load, from its initial position at the start of the test. In addition to measuring the deflection of the surface of the pavement, a method was developed by means of which the deflection at the top of the base course and at the top of the subgrade could be recorded simultaneously. This is accomplished by measurements made with vertical rods which pass through two holes, each 1½ inches from the center of the bearing plates, and into matching holes drilled to the proper depths in the pavement structure (see fig. 4). To eliminate friction between the vertical rods and the materials through which they pass and to provide some lateral support for these materials, metal tubes or casings are inserted in the drilled holes. Care is taken to provide clearance between the lower end of the tube and the seat of the deflection rod in order that any compression of the upper layers which might develop during the test would not bring the tubes in contact with the deflection rod seats. These seats consist of a hardened plaster plug or bulb formed by extruding soft plaster through the tubes by means of a snugly fitting plunger. This method was developed to overcome the tendency of the material at the base of the holes, particularly the subgrade, to intrude into the openings during the test.

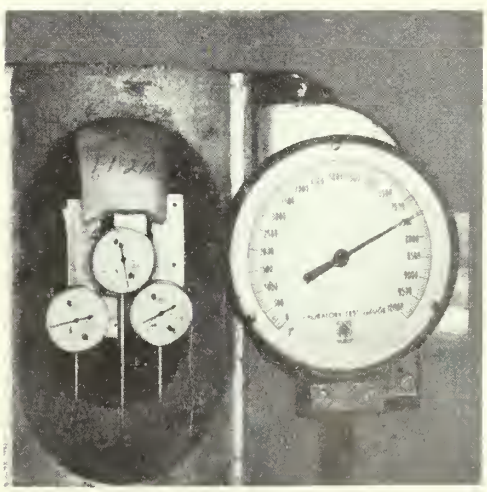


Figure 6.—Typical photographic record of deflection dials and load-pressure gage.

A photograph of a typical load set-up with the 30-inch diameter bearing plate is shown in figure 5. The 24-, 18-, and 12-inch plates are pyramided on the 30-inch plate for rigidity. Above the bearing plates is a spherical bearing surface. Resting on this surface is a cut-out cylindrical section in which the special 2-inch range-deflection measuring dials are suspended by arms extending from the deflection or dial-support beam. The cover at the upper end of the cylindrical section supports a 50-ton hydraulic ram. Supplying oil to the ram is a high-pressure, low-volume, radial-type hydraulic pump driven by an electric

motor through a variable-speed drive. The maximum operating pressure of the pump is about 10,000 p.s.i., providing the maximum reaction of about 110,000 pounds. A hand pump connected into the line to the ram has been found useful in maintaining the load increments required in the first part of the test procedure.

The hydraulic pressure gage is mounted near the deflection dials so that all readings may be photographed at intervals during the continuous test, to failure. The camera is a very compact 35 mm. spring-motor-powered automatic type. It is not a moving-picture camera, but takes a picture and rolls up the film ready for the next exposure each time a button is pressed. During the progress of a test the operator takes a picture at each 0.1 inch of plate deflection as indicated by the measuring dial. After the film has been developed the negative is projected with a strip-film projector on a small screen from which readings of the simultaneous values of load, deflection of pavement surface, deflection of base course, and deflection of subgrade are taken. A sample frame taken from a film record is reproduced in figure 6.

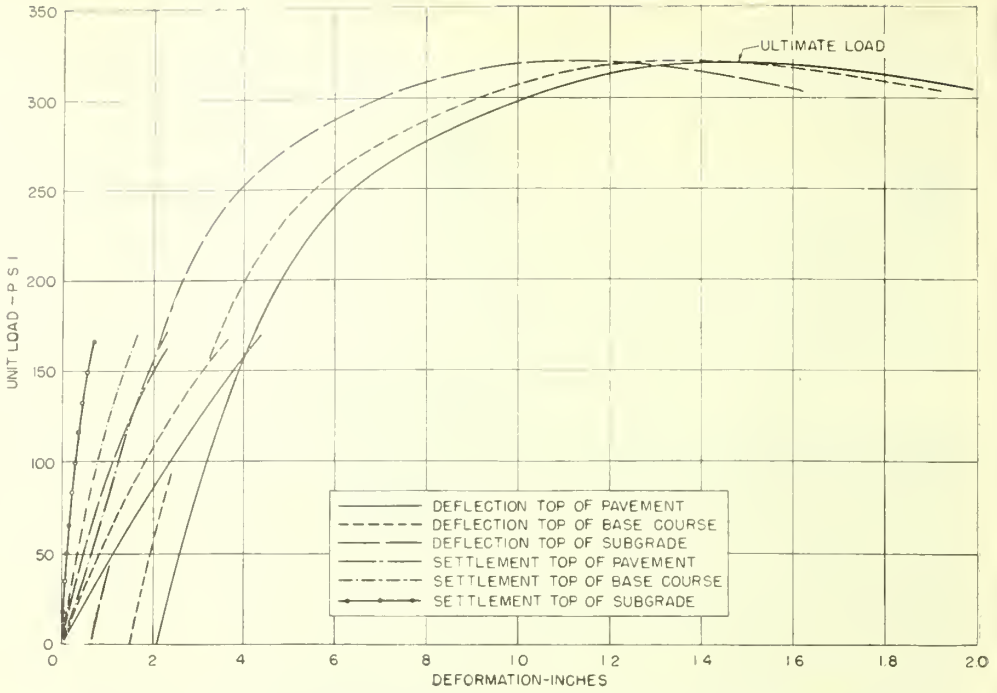


Figure 7.—Typical load-deflection and load-settlement data: 1948 procedure.

TYPICAL TEST DATA

Typical data resulting from this test procedure are shown in figure 7. The shorter lines radiating from the origin represent the first part of the test in which the load is applied and released in three increments. Three of these lines are the load-deflection relations for the three components of pavement structure; that is, the surfacing, base, and subgrade. The other three are the load-settlement relations of the same three components of pavement structure. The remaining three curves that sweep across the graph are the result of the latter part of the test, in which the load is applied continuously until a deflection of 2.0 inches is reached. These are load-deflection relationships starting from the last settlement values obtained for the surface, base, and subgrade, respectively. It is to be noted that an ultimate load value is reached, after which the curves descend slightly. As stated before, in this type of test the load is applied in such manner as to give a constant deflection rate of 0.5 inch per minute.

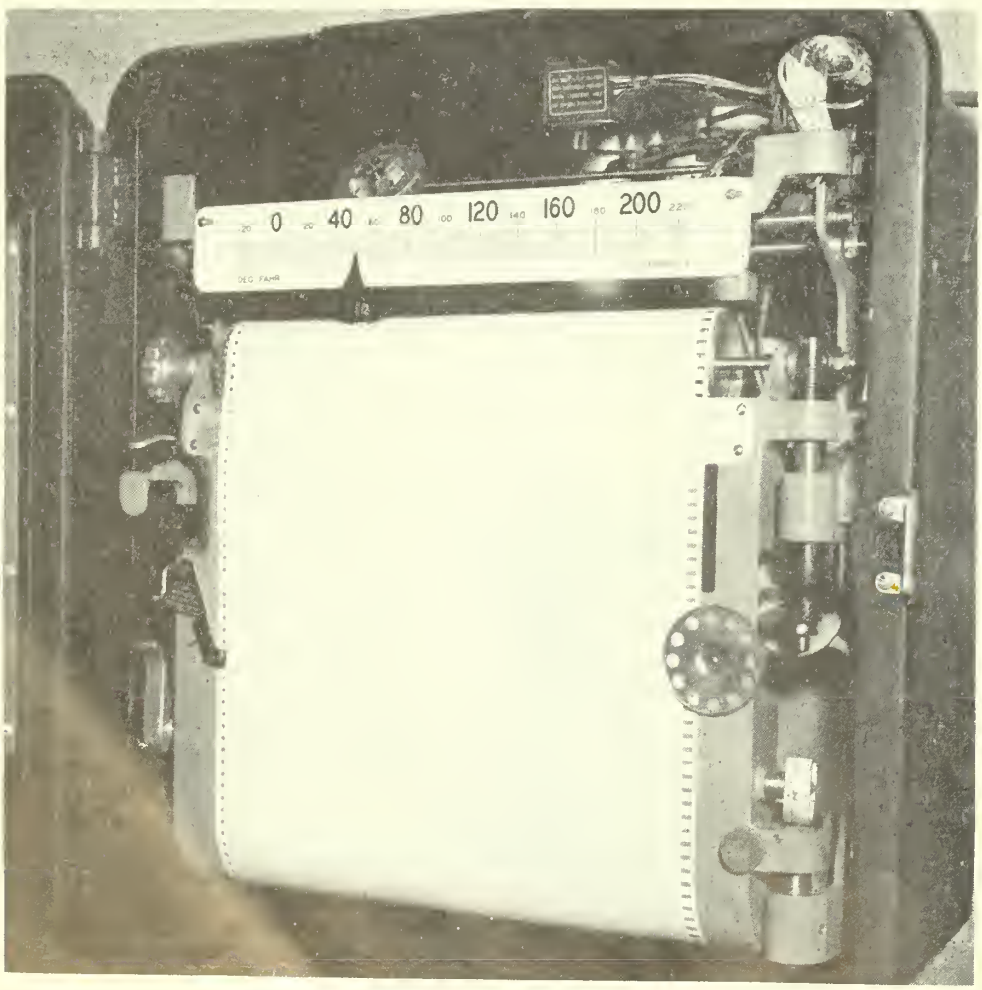


Figure 8.—Thermocouple temperature recorder.

In addition to load testing the surface of the pavement, tests are also made directly on the base, with the surfacing removed, and on the subgrade with the total depth of pavement removed. All tests are made in duplicate.

Tests on the base course and subgrade are made in two ways: one, with surcharge, where the opening in the overlying pavement components is of a size just sufficient to accommodate the plate; the other, without surcharge, where the diameter of the opening is three times the diameter of the plate.

Using the equipment and procedure described, over 200 individual loading tests were made during the summer of 1948, an average of about four tests per 8-hour day.

SPECIAL SUBGRADE BEARING TESTS

In an investigation of this nature it is considered essential to develop as complete knowledge as possible of bearing capacity of the subgrade soil, particularly as influenced by the size of loaded area. Obviously the character of this relation will depend upon the degree of uniformity of the material. A soil mass uniform with respect to composition,

(*Continued on page 200*)

Life Characteristics of Highway Surfaces

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH, BUREAU OF PUBLIC ROADS

Reported[1] by FRED B. FARRELL, Chief, Highway Cost Section and HENRY R. PATERICK, Head, Annual Cost Unit

One objective of the road-life study phase of the highway planning surveys is the analysis of service-life characteristics of primary rural highway surfaces. The first comprehensive report on this subject was published in PUBLIC ROADS in 1941. The present article reports further information and compares the original service-life estimates with actual later experience.

The reported analyses are based on 248,783 miles of construction and 129,593 miles of retirement, up to 1946, on rural State or Federal-aid primary systems in 16 States. Estimates of average service lives range from 4½ years for low-type to 27 years for high-type surfaces.

For 12 States represented in both reports, future mileages remaining in service were estimated from the average life analyses in the 1941 study, based on actual experience to 1937. Five-year forecasts compared closely with actual mileages remaining in service in 1942. Greater differences were found between the ten-year forecasts and actual mileages remaining in service in 1946, probably due to the lagging construction program during the war.

On January 1, 1946, the average age of low-type surfaces was 10.6 years; of intermediate types, 8.2 years; and of high types, 13.5 years. Corresponding remaining life expectancies of the mileages in service were 4.5, 7.6, and 12.0 years. Of the mileages in service in 1946, it is estimated that 94 percent of the low-type, 74 percent of the intermediate-type, and 44 percent of the high-type surfaces will require rebuilding by 1956.

Of 92,565 miles of retirements through 1945, 58 percent was resurfaced, 30 percent reconstructed, 10 percent transferred to other public agencies, and 2 percent abandoned. During the war the proportion of resurfacing of high types increased noticeably.

Preliminary studies indicate that, because of salvage value at retirement, the average dollar investment life exceeds the average service life of a surface on a mileage basis. For gravel surfaces constructed in 1920–25 in one State, the average service life of the surface was 5.6 years, while the dollar investment life was 8.6 years.

THE PRESENT STAGE of development of the Nation's highways, roads, and streets is the result of tremendous accomplishments over the past 30 years. Proper management of this vast transportation system requires the collection, analysis, and interpretation of complete factual data relating to the construction, maintenance, operation, and administration of the highway plant. The highway planning surveys, undertaken in the middle 1930's by the State highway departments in cooperation with the Bureau of Public Roads, were conceived for this purpose.

The highway plant itself is the proving ground from which can be obtained the facts relating to past and present performance of various types of highway construction under varying degrees of use. An evaluation of these facts in relation to known conditions is essential in order that plans for orderly future development can be undertaken with confidence. The road-life study phase of the comprehensive highway planning surveys provides the means for obtaining the data upon which such evaluations can be made. Among its objectives is the determination of the annual cost of highways, embracing such items as rates of wearing out, construction costs, maintenance costs, extent of functional obsolescence and structural deterioration, and life of the investment.

Up to the present time, the road-life studies in the individual States have been confined largely to the primary State highway systems or the Federal-aid primary systems. They are gradually being extended to State secondary and Federal-aid secondary systems and to city streets.

The principal progress in this field of research has been in the determination of the life characteristics of highway surfaces. The first comprehensive report on this subject was made in 1941.[2] In that report (hereafter referred to simply as the 1941 report) were included the results of service-life analyses of 210,000 miles of construction up to January 1, 1937, of various surface types in 26 States.

Activity on the road-life study was considerably curtailed during World War II, and the work was not resumed on a broad scale until 1946. Of the 37 States which now have prepared road-life mileage tabulations, there are 16 with a cut-off date of January 1, 1946. The tabulations for the remaining 21 States are not sufficiently current to warrant inclusion in the present analysis.

Data for the following 16 States are included in this report:

Idaho	Nevada
Illinois	New Mexico
Indiana	Oklahoma
Kansas	Texas
Maryland	Utah
Minnesota	West Virginia
Missouri	Wisconsin
Montana	Wyoming

BASIC DATA COMPILED

The basic data compiled for the purpose of this report embrace 248,783 miles of construction of various surface types on the rural portions of the primary State or Federal-aid systems of these 16 States. In general, all mileage in incorporated places of greater than 1,000 persons has been excluded. Construction of widening has also been omitted in those cases where the widening was done as a separate operation. Where the widening was done in conjunction with the resurfacing or reconstruction of the previous surface, the mileage of the new construction, which includes widening, is represented in the tabulations. The basic summaries cover the period from January 1, 1900, to January 1, 1946, and include:

1. Mileage constructed each year for each surface type (for 16 States).

2. Mileage of each year's construction of each surface type remaining in service on January 1 each year after construction (for 16 States).

[1] Presented at the 28th annual meeting of the Highway Research Board, December 1948.

[2] *Life characteristics of surfaces constructed on primary rural highways*, by Robley Winfrey and Fred B. Farrell; PUBLIC ROADS, vol. 22, No. 1, March 1941.

3. Replacement surface types for mileage of each surface type retired each year (for 13 States).

4. Method of retirement (resurfaced, reconstructed, abandoned, or transferred) for mileage of each surface type retired each year (for 13 States).

Data for Idaho, Wisconsin, and Wyoming were not available for the summaries prepared in connection with items 3 and 4.

In addition, there are certain data from three of the States—Missouri, West Virginia, and Wisconsin—relating to dollars invested each year and remaining in service each subsequent year. Some results of these analyses are also included in this report.

There are eight major surface types for which individual service-life analyses were made:

1. Soil surfaced.
2. Gravel or stone.
3. Bituminous surface treated.
4. Mixed bituminous.
5. Bituminous penetration.
6. Bituminous concrete.[3]
7. Portland-cement concrete.
8. Brick or block.

Definitions of these surface types, used in all phases of the highway planning surveys in determining the general type classifications constructed in the individual States, will be found in the appendix, page 196. Definitions of the four methods of retirement will also be found in the appendix.

AVERAGE LIFE DEFINED

The average service life of a road surface is the average period of time after construction that the surface remains in service prior to being replaced, resurfaced, reconstructed, or otherwise taken out of service for any reason or by any method. Stated in another manner, it is that period of time after construction during which the only operations performed on the road surface are those of maintenance as practiced by the various States. It is a recognized fact, however, that a significant amount of construction work is done by maintenance forces and paid for out of maintenance funds in many States. In recording the original data summarized in this report, an attempt was made in each State to segregate construction from maintenance in a uniform manner regardless of the particular accounting practices in effect in a given State. The classifications of construction and maintenance operations generally followed in the road-life study are those included in the tentative draft of the report to the 1938 meeting of the Subcommittee on Uniform Accounting of the American Association of State Highway Officials.[4]

[3] Including sheet asphalt and rock asphalt.

ANALYSIS PROCEDURES

The survivor curve analysis procedures employed in this report are the same as those which are discussed at some length in the 1941 report. Reference should be made to this earlier report for an explanation of the mechanics of computing average service life. There is one difference in the manner of analyzing the data compiled by the States: In the 1941 report, a single analysis was made of the combined data for all States; in the present report, individual analyses were made for each State and the results combined by weighting. The two procedures will yield the same result, however.

The average life data included in this report represent estimates based upon actual experience. Over the years there have been changes in construction methods and design standards. There have been periods of accelerated activity and periods when little or no construction work was accomplished. Some roads have been kept in service too long while others have been rebuilt before the end of their useful life. Maintenance has frequently been inadequate. There have been many instances of overdesign and underdesign.

Throughout the past 30 years, nevertheless, there has been sustained improvement in the standards of highway design, construction, maintenance, and administration. Each of these has its influence upon service life, but their individual effects cannot be evaluated with certainty. As a result of improvements which are continually being made in design standards, for example, certain factors contributing to early obsolescence or structural failure are gradually being reduced to a minimum, or even eliminated.

[4] Copies of this tentative draft were transmitted to all State highway departments under date of June 2, 1938, by the Subcommittee on Uniform Accounting, American Association of State Highway Officials.

It must be recognized, also, that large backlogs of needed replacements of highway facilities have been accumulating for many years. The extent of these backlogs is forcibly brought to attention in the long-range highway needs studies which are under way or which have been recently completed in several States. If the accumulated deficiencies in the highway plant are to be overcome at the rates recommended in these long-range studies, it is likely that the probable remaining service lives may, in some instances, prove to be somewhat less than indicated by the data presented in this report.

While it is true that substantial advances are being made in obtaining greater value from the investment in highways, it should not be concluded that the highway plant can or should be built to last indefinitely. There has been in the past and will continue to be in the future a need for all types of highway facilities. The complexities of modern transportation will also result in shifts in the importance of various roads and streets. Traffic patterns are subject to continual change, and the highway plant must be sufficiently flexible to absorb these changes.

In actual practice only a small percentage of road sections have a life exactly equal to the average. Thus, when there is need for an estimate of service life for a particular road section it is necessary to consider such factors as age, structural condition, design features, location, and traffic usage which relate to that section. Only by the exercise of expert engineering judgment in the evaluation of these factors is it possible to arrive at an estimate of the remaining service life for a particular road section.

MILEAGE IN SERVICE

In table 1 is listed for each surface type, by 5-year construction periods, the mileage constructed during each period and the mileage remaining in service on January 1, 1946. Approximately 38 percent of the surfaced mileage on the primary rural State highway systems is represented by these data, the proportion of each surface type being as follows (*see next page.*):

Table 1.—*Mileages constructed and mileages remaining in service on Jan. 1, 1946, for each surface type* [1]

Construction year period	Soil surfaced		Gravel or stone		Bituminous surface treated		Mixed bituminous		Bituminous penetration		Bituminous concrete		Portland-cement concrete		Brick or block	
	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service
1905 and prior	-	-	36.0	0.0	-	-	-	-	-	-	-	-	-	-	0.3	0.0
1906–10	-	-	198.7	0.0	-	-	-	-	4.4	0.0	-	-	1.0	0.0	3.7	0.0
1911–15	34.9	0.0	4,330.2	45.8	18.3	9.4	3.7	2.4	29.2	4.2	24.8	7.1	398.5	137.8	65.2	16.4
1916–20	160.0	0.0	6,307.1	146.3	202.1	60.9	182.1	126.4	231.0	42.2	205.8	57.0	1,457.9	654.4	153.6	30.0
1921–25	436.3	7.6	21,521.4	1,194.9	618.7	120.0	1,043.1	640.8	1,290.2	371.6	821.6	297.3	8,854.9	6,651.4	331.2	139.8
1926–30	146.9	18.5	23,176.3	2,177.9	4,104.7	984.6	5,801.0	1,966.1	1,850.9	824.9	1,259.6	559.2	12,486.8	11,637.1	135.1	98.7
1931–35	667.9	48.2	18,998.6	3,376.5	9,300.6	2,782.1	22,470.4	10,970.9	2,886.4	2,041.5	1,521.4	1,014.7	10,581.1	10,331.8	161.8	97.1
1936–40	1,364.7	125.5	10,911.2	3,388.6	12,155.7	8,167.9	22,997.9	15,062.9	1,452.6	1,206.8	2,928.2	2,365.3	4,459.6	4,408.8	23.4	20.4
1941–45	588.5	165.6	4,913.0	3,348.8	8,487.4	7,819.9	7,561.4	7,082.6	1,032.4	1,004.6	3,742.8	3,686.2	1,634.4	1,628.1	14.5	14.5
Total	[2] 399.2	365.4	90,392.5	13,678.8	34,887.5	19,944.8	60,059.6	35,852.1	8,777.1	5,495.8	10,504.2	7,986.8	39,874.2	35,449.4	888.8	416.9

[1] Compiled from data submitted by 16 States for rural State or Federal-aid primary systems.

	Percent
Soil surfaced	13
Gravel or stone	27
Bituminous surface treated	32
Mixed bituminous	54
Bituminous penetration	23
Bituminous concrete	32
Portland-cement concrete	44
Brick or block	31
Average, all types	38

There are some mileages, particularly of the lower types, for which the date of retirement is known but for which the date of initial construction was not available. This results primarily from the difficulty in locating records of early construction. The partial data in these cases are not included in the analysis.

The probable average service lives for each surface type are shown in table 2, for 5-year construction periods. Estimates of average lives are given in this table only when retirements were sufficient to warrant making the estimates. Because of the smaller mileages involved, the retirement trends for earlier construction are frequently more erratic than the trends for the larger mileages of more recent construction. The average life estimates for this earlier construction are more reliable, however, because of the greater retirement experience.

In addition to the major types listed in table 2, there are certain subtypes for which the average service lives have been computed. Probably the most interesting of these subtypes is old portland-cement concrete which has been resurfaced with mixed bituminous or bituminous concrete surfaces. Of the 2,500 miles of this subtype included with the major type in this report, 77 percent involves old portland-cement concrete resurfaced with bituminous concrete and 23 percent resurfaced with mixed bituminous surfaces. For the major mileage of this subtype, there have been insufficient retirements upon which to base an estimate of average life. During the period 1926–35, however, there were about 230 miles of old concrete resurfaced, of which less than 10 percent was still in service on January 1, 1946. On the basis of this retirement experience, the additional service life obtained from old portland-cement concrete roads after they are resurfaced is 13.7 years. The limited retirement experience for later construction of this subtype indicates that the average service life will remain in the neighborhood of 14 years.

INFLUENCE OF WAR ON RETIREMENTS

The war period has had an influence upon the retirement trends of various surface types. Many surfaces were kept in service beyond their normal life even though subjected, in many cases, to greater wear than at any previous period. In order to reflect the additional life thus obtained in the average life estimates included in this report, it was necessary to make a minor adjustment to the analysis procedures outlined in the 1941 report. This was accomplished by assuming that the backlog of deferred work during the war would be overcome and the normal trend resumed within a maximum of 10 years after the war. The rate of recovery estimated in each instance depended upon the percentage surviving at the beginning of the war, the age of these survivors, the previous rate of retirement, and the retirements, if any, which were made during the war period.

Table 2.—*Weighted probable average service lives for various construction-year periods for each surface type* [1]

Construction-year period	Surface type							
	Soil surfaced	Gravel or stone	Bituminous surface treated	Mixed bituminous	Bituminous penetration	Bituminous concrete	Portland-cement concrete	Brick or block
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
1905 and prior	----	19.9	-------	-------	-------	-------	-------	40.0
1906–10	-------	12.0	-------	-------	15.6	-------	27.7	24.7
1911–15	6.3	14.7	23.9	24.4	14.8	24.9	23.9	24.7
1916–20	12.1	10.5	17.9	25.4	15.3	18.3	24.5	21.6
1921–25	8.7	9.1	14.4	21.9	16.2	18.1	26.1	20.2
1926–30	7.2	6.6	9.1	12.3	16.0	15.6	28.2	21.1
1931–35	4.5	5.9	7.4	11.6	15.9	14.7	-------	16.4
1936–40	2.9	5.6	10.3	11.6	14.5	13.5	-------	-------
1941–45	2.1	5.1	-------	-------	-------	-------	-------	-------

[1] Based on analyses of data submitted by 16 States for rural State or Federal-aid primary systems. Average lives shown in this table are to the nearest 0.1 year, but they should not be presumed accurate to this extent. The averages would be materially affected by excluding certain States or by including additional States.

No particular significance should be attributed to the differences in average lives shown in table 3. Variations in average lives of this magnitude are not uncommon when the analyses are based upon different groupings of States. Further, unusual construction practices in one State may have considerable effect upon the average for a group of States. In Wisconsin, for example, the practice of frequent resurfacing of bituminous surface-treated and mixed-bituminous roads has been

COMPARISON OF TWO STUDIES

There are certain differences between the average lives presented in the 1941 report and those listed in the present report. Table 3 shows a comparison of the average lives for various types for the most recent 5-year periods for which data were listed in the 1941 report.

Table 3.—*Comparison of average lives presented in the 1941 report with those listed in the present report*

Surface type	Comparison period [1]	1941 report (26 States)		Present report (16 States)	
		Miles constructed	Average life	Miles constructed	Average life
		Miles	*Years*	*Miles*	*Years*
Soil surfaced	1931–35	[2] 2,542	[2] 5.4	668	4.5
Gravel or stone	1931–35	22,793	6.0	18,999	5.9
Bituminous surface treated	1931–35	[2] 10,286	[2] 11.4	9,301	[3] 7.4
Mixed bituminous	1926–30	5,610	14.3	5,801	[4] 12.3
Bituminous penetration	1926–30	3,725	17.0	1,851	16.0
Bituminous concrete	1921–25	2,362	17.9	822	18.1
Portland-cement concrete	1921–25	6,737	24.4	8,855	26.1
Brick or block	1921–25	980	18.2	331	20.2

[1] The most recent period for which data were presented in table 18 in the 1941 report.
[2] 1931–34.
[3] If Wisconsin data are excluded, average life is 10.2 years.
[4] If Wisconsin data are excluded, average life is 13.0 years.

Table 4.—*Comparison of forecasts of the amounts of surfacing remaining in service with later actual experience* [1]

Surface type	Original mileage constructed [2]	Percentage of original construction remaining in service			
		As of Jan. 1, 1942		As of Jan. 1, 1946	
		Forecast	Actual experience	Forecast	Actual experience
	Miles	*Percent*	*Percent*	*Percent*	*Percent*
Soil surfaced	781	11	22	2	8
Gravel or stone	40,416	12	14	4	11
Bituminous surface treated	7,420	51	52	34	40
Mixed bituminous	7,449	56	55	37	49
Bituminous penetration	3,533	53	55	37	37
Bituminous concrete	989	45	46	27	38
Portland-cement concrete	3,471	63	69	48	60
Brick or block	408	58	56	42	34
Total	64,467	27	29	16	23

[1] Comparison of data for 12 States included in both the 1941 report and the present report. Forecasts were based on analyses of previous tabulations which showed actual experience to Jan. 1, 1937.
[2] Includes mileage constructed through the latest year for which average life data were presented in the 1941 report.

Table 5.—*Mileages retired for which methods of retirement were determined* [1]

Surface type	Miles retired during each year group						
	1927 and prior	1928–30	1931–33	1934–36	1937–39	1940–42	1943–45
Soil surfaced	88.9	77.2	256.8	308.9	697.2	907.6	431.4
Gravel or stone	4,335.9	8,310.7	14,459.3	12,227.0	9,552.3	6,950.9	1,979.8
Bituminous surface treated	37.0	136.3	802.6	1,488.4	2,435.8	2,240.5	2,705.5
Mixed bituminous	31.2	153.0	832.3	1,448.2	4,603.0	2,759.9	2,455.4
Bituminous penetration	137.5	145.7	271.5	489.5	433.1	689.8	1,023.4
Bituminous concrete	51.1	39.7	170.5	298.8	435.0	615.3	852.1
Portland-cement concrete	109.7	151.6	369.7	347.4	592.4	690.8	1,464.9
Brick or block	18.6	27.0	56.2	35.7	84.1	107.8	143.1
Total	4,809.9	9,041.2	17,218.9	16,643.9	18,832.9	14,962.6	11,055.6

[1] Compiled from data submitted by 13 States for rural State or Federal-aid primary systems.

Table 6.—*Retired mileages for each surface type, and percentage distribution according to method of retirement (total for 1945 and prior)* [1]

Surface type retired	Total retired	Methods of retirement				
		Resurfaced	Reconstructed	Abandoned	Transferred	Total
	Miles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Soil surfaced	2,768.0	68.6	24.5	1.8	5.1	100.0
Gravel or stone	57,815.9	58.8	29.7	2.3	9.2	100.0
Bituminous surface treated	9,846.1	58.9	27.1	3.4	10.6	100.0
Mixed bituminous	12,283.0	51.3	37.3	2.0	9.4	100.0
Bituminous penetration	3,190.5	56.3	24.7	3.7	15.3	100.0
Bituminous concrete	2,462.5	65.9	19.8	2.4	11.9	100.0
Portland-cement concrete	3,726.5	54.8	26.6	3.3	15.3	100.0
Brick or block	472.5	53.3	30.6	2.7	13.4	100.0
Total	92,565.0	58.0	29.8	2.4	9.8	100.0

[1] Computed from data submitted by 13 States for rural State or Federal-aid primary systems.

followed. If the experience for Wisconsin is excluded from the analyses for the 16 States included in this report, the average lives for these types show a closer agreement with the 1941 report.[5]

There are 12 States [6] included in the present report which were included in the 1941 report. For these 12 States it is now possible to compare forecasts of mileages remaining in service, as obtained from analyses made in 1941, with actual experience. In the 1941 report, estimated average lives for the various surface types were presented up to and including the most recent year for which the retirement experience was sufficient to enable a reasonable estimate of the average life to be made. Based upon these previous estimates of average lives, projections were made, for each of the 12 States, of the probable mileages of each surface type that would still be in service on January 1, 1942, and on January 1, 1946. The extent to which these forecasts varied from the actual experience is shown in table 4.

For the most part, the forecasted amounts and the actual amounts remaining in service on January 1, 1942, show reasonable agreement. The greatest difference is in the soil-surfaced type but, because of its short life and limited mileage, its retirement trend is usually erratic. For the other types, the forecasts and the actual amounts remaining in service are in close agreement except for portland-cement concrete where the retirements were somewhat overestimated.

Among the probable reasons for the differences are a lagging highway program and the possible tendency to underestimate the average life upon which the forecast was based.[7] There are doubtless other reasons involved, but their specific influence on the previous analyses are most difficult to determine and hence have not been evaluated.

Greater differences are apparent in the comparison of forecasts and actual mileages remaining in service on January 1, 1946. In this case, the differences can be attributed to a large extent to the lagging highway program during the war period.

The evidence in table 4 would indicate that the analysis procedures employed in the 1941 report yielded generally satisfactory results when appraised in the light of actual experience. With more actual experience in construction and retirements to draw upon, with each successive year, it can be expected that the analysis techniques will yield results progressively more reliable.

[5] The data for Wisconsin were not included in the 1941 report.

[6] Idaho, Indiana, Kansas, Maryland, Missouri, Montana, New Mexico, Oklahoma, Texas, Utah, West Virginia, and Wyoming.

[7] In Bulletin 125 of the Iowa Engineering Experiment Station, *Statistical Analyses of Industrial Property Retirements*, by Robley Winfrey, pp. 86–92, the probable error in determining average life by comparison with the 18 type survivor curves is discussed. In this connection tests were made upon the reliability of the average life estimates, which indicate two significant results: ". . . . First, the average of the under- and over-estimates of probable average life balances close to a zero error; second, the average negative error and the average positive error are about 10 percent at the 70 percent-surviving level and about 15 percent at the 80 percent-surviving level. For certain short curves, however, individual estimates of probable average lives may be greatly in error when compared with the average life determined from the completed curve. Such large errors are caused mainly by two factors: First, often the few data in the upper 10 to 30 percent of the survivor curves are insufficient to definitely fix the curve through this area especially when these few points do not follow a smooth path; second, the succeeding data frequently depart from the trend established by the first few data."

Table 7.—*Percentage retired by various methods for all surface types combined, for various periods* [1]

Method of retirement	Period							
	1927 and prior	1928–30	1931–33	1934–36	1937–39	1940–42	1943–45	Total, 1945 and prior
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Resurfaced	65.2	57.8	51.3	60.4	60.4	52.4	65.7	58.0
Reconstructed	24.3	30.7	34.5	26.0	27.5	35.2	26.0	29.8
Abandoned	1.1	1.1	2.9	2.8	3.1	2.7	1.1	2.4
Transferred	9.4	10.4	11.3	10.8	9.0	9.7	7.2	9.8
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Computed from data submitted by 13 States for rural State or Federal-aid primary systems.

Table 8.—*Percentage of total retirements by resurfacing for various types for various periods* [1]

Surface type retired	Period							
	1927 and prior	1928–30	1931–33	1934–36	1937–39	1940–42	1943–45	Total, 1945 and prior
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Soil surfaced	83.5	38.3	51.3	75.7	82.3	68.3	54.6	68.6
Gravel or stone	65.8	58.8	51.9	66.7	59.8	56.3	49.6	58.8
Bituminous surface treated	63.2	49.2	52.2	51.7	64.8	53.8	64.4	58.9
Mixed bituminous	51.3	52.4	41.5	33.4	59.8	39.9	62.0	51.3
Bituminous penetration	68.8	47.8	30.7	32.7	46.7	49.8	82.2	56.3
Bituminous concrete	4.5	32.2	70.9	38.5	67.2	56.8	85.8	65.9
Portland-cement concrete	59.5	50.5	54.9	40.1	38.5	36.4	73.6	54.8
Brick or block	26.9	14.5	28.5	18.8	37.3	55.1	90.4	53.3
Total	65.2	57.8	51.5	60.4	60.4	52.4	65.7	58.0

[1] Computed from data submitted by 13 States for rural State or Federal-aid primary systems.

CLASSIFICATION BY RETIREMENT METHOD

In 13 of the 16 States included in the average life analyses, the retired mileages for various years were classified in accordance with the method by which the retirement was made. These retired mileages are listed in table 5.

The methods of retirement into which these mileages were classified are (1) resurfaced, (2) reconstructed, (3) abandoned, and (4) transferred. Definitions of these four general methods of retirement will be found in the appendix, page 196.

Reversion in type, in which a surface reverts to a lower type through lack of adequate maintenance, also represents a distinct method of retirement. Retirements of this nature were so few that they did not warrant consideration in this analysis.

These retirement classifications are general in character and should be so interpreted. Resurfacing is an especially significant method of retirement since it affords an approximate measure of the relative extent to which the various types of surfacing construction are salvaged when they are retired.

In table 6 are shown the total mileages retired and the percentage distribution by methods of retirement. For each surface type, the method of retirement for more than half of all retirements has been by resurfacing. The over-all relationships shown in table 6 have been fairly consistent throughout the years, as is evident in table 7.

RESURFACING INCREASED IN 1943–45

These higher percentages of resurfacing during the period 1943–45 are due in large measure to the curtailed highway program during the war years. Undoubtedly, many miles that ordinarily would have been reconstructed were given a resurfacing treatment to keep them in operation temporarily. Other mileages which would normally have been replaced because of poor location were also kept in service until they could be rebuilt at some future date. Both of these factors unquestionably contributed to reducing the amount of reconstruction and thereby increased the relative amount of resurfacing during this period.

Table 8 shows for each surface type the amounts of resurfacing for various periods of years. It is most interesting to note that the relative amount of resurfacing for mixed bituminous and higher type surfaces was at its greatest value during the period 1943–45. For portland-cement concrete the percentage during this period was double that of the preceding 3 years.

Table 9.—*Mileage in service on Jan. 1, 1946, and estimated percentages which will remain in various future years* [1]

Surface type	In service on Jan. 1, 1946	Remaining in service on			
		Jan. 1, 1951	Jan. 1, 1956	Jan. 1, 1961	Jan. 1, 1966
	Miles	*Percent*	*Percent*	*Percent*	*Percent*
Low	14,044.2	31.2	6.5	1.1	0.1
Intermediate	55,795.9	58.9	26.0	8.3	2.0
High	49,348.9	80.1	56.0	32.8	15.1
Total	119,190.0	64.4	36.1	17.6	7.2

[1] Based on analyses of data submitted by 16 States for rural State or Federal-aid primary systems.

ANALYSIS BY MAJOR TYPE GROUPS

Figures 1 and 2 show, in graphical form, some of the results of the analyses presented in the preceding pages. For purposes of simplifying these charts, the eight surface types have been combined into three major groups—low, intermediate, and high—as follows:

Low type: Includes soil-surfaced and gravel or stone roads.

Intermediate type: Includes bituminous surface-treated and mixed bituminous roads.

High type: Includes bituminous penetration, bituminous concrete, portland-cement concrete, and brick or block roads.

These combinations are arbitrary and may be criticized in certain respects. Some of the mixed bituminous roads, for example, qualify as high type since they are placed on old rigid bases or on heavy flexible bases of considerable load-bearing capacity. On the other hand, some mileages of the high types are in the intermediate-type category. However, the total mileage of questionable classification is small and considerable effort would be required to insure that each mile was properly classified. Accordingly, no adjustments in this respect were made since the increase in accuracy obtained would not be sufficient to justify such refinement.

Estimates of service lives based upon actual retirement experience were shown in table 2. In order to prepare figures 1 and 2 certain assumptions of average lives are necessary for the recent years. In general, the average lives for the more recent years were assumed to remain about the same or increase slightly (less than 10 percent) in relation to the estimates for the most recent years shown in table 2. These assumptions will no doubt vary somewhat from the actual future experience, but since the bulk of the retirements within the next few years will come from the older construction, any minor differences from the assumed average lives for the more recent construction will not have any major effect upon the over-all trends which are shown.

TRENDS IN SERVICE LIFE

Figure 1 shows, for each 5-year construction period, the trend of mileages of low, intermediate, and high types in service up to January 1, 1946, for the 16 States included in this report, and the rates at which these mileages will go out of service in the future based upon their probable rates of retirement. Table 9 shows the total mileages in service on January 1, 1946, and the probable amounts which will still remain in service for 5, 10, 15, and 20 years in the future.

Table 2 showed the probable average lives for various construction-year periods. These average lives are the expectancies at the time of construction or at age zero. As the road system develops and becomes older, the average age of the surfaces increases and the remaining life expectancy becomes less. Also, as the system becomes older, mileages of earlier retirements are taken out of service, thus leaving in service those mileages whose lives will exceed the average life of the total original construction.

LIFE EXPECTANCY

The probable life of the mileage in service is equivalent to the age plus the expectancy. Under certain conditions it is possible for the average age of mileage in service to exceed the average lives shown in table 2. This is true for low-type surfaced mileage, from which the miles in service have gradually been diminishing since 1931 (see fig. 1). When these low types have been retired, the replacement types have been intermediate or high types. Thus, there has not been sufficient construction of new low types to keep the average age of all low types in service from increasing year to year. As a result, the low types now in service are quite old and have a short remaining life expectancy.

In table 10 are shown the average age, remaining life expectancy, and total probable life of the mileages of low, intermediate, and high types in service, at 5-year intervals from January 1, 1921, to January 1, 1946. This information is also presented graphically in figure 2, and shows strikingly the trends in the

Table 10.—*Average age, life expectancy, and probable life of mileages in service at 5-year intervals* [1]

Date	Low types			Intermediate types			High types		
	Age	Expectancy	Probable life	Age	Expectancy	Probable life	Age	Expectancy	Probable life
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
Jan. 1, 1921	4.2	8.6	12.8	1.9	19.9	21.8	2.6	20.2	22.8
Jan. 1, 1926	4.0	7.0	11.0	3.5	16.5	20.0	3.2	21.1	24.3
Jan. 1, 1931	5.3	5.3	10.6	2.7	11.1	13.8	4.9	20.4	25.3
Jan. 1, 1936	6.1	5.8	11.9	3.7	10.1	13.8	7.3	18.0	25.3
Jan. 1, 1941	7.9	6.7	14.6	5.2	10.1	15.3	10.4	14.9	25.3
Jan. 1, 1946	10.6	4.5	15.1	8.2	7.6	15.8	13.5	12.0	25.5

[1] Based on analyses of data submitted by 16 States for rural State or Federal-aid primary systems.

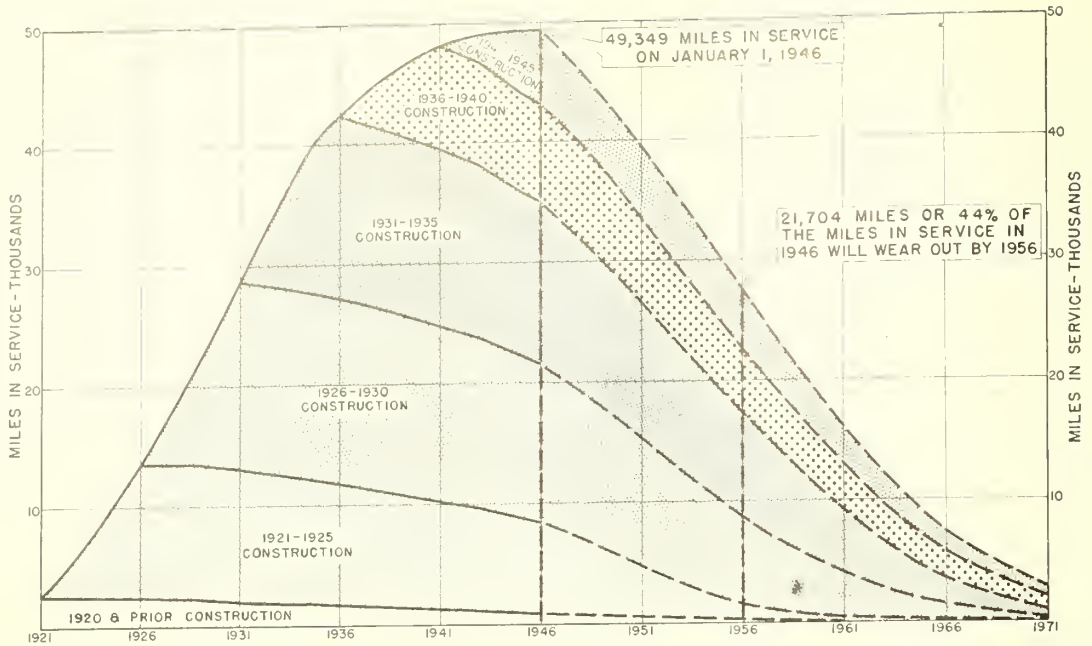
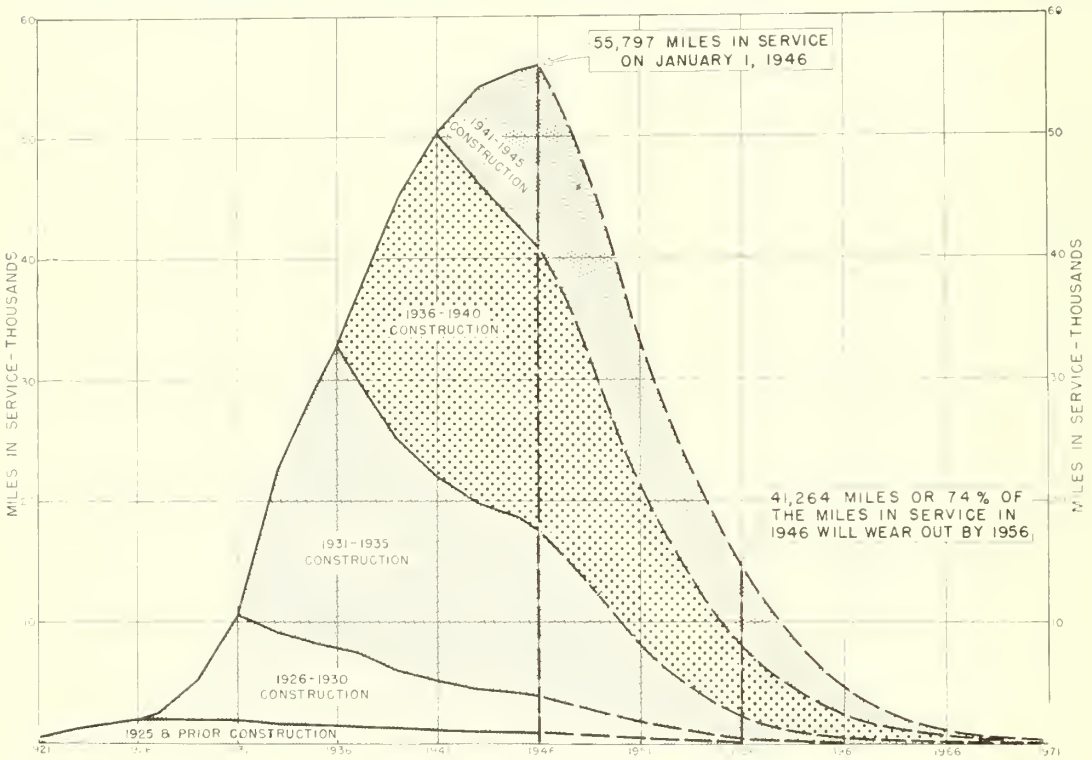
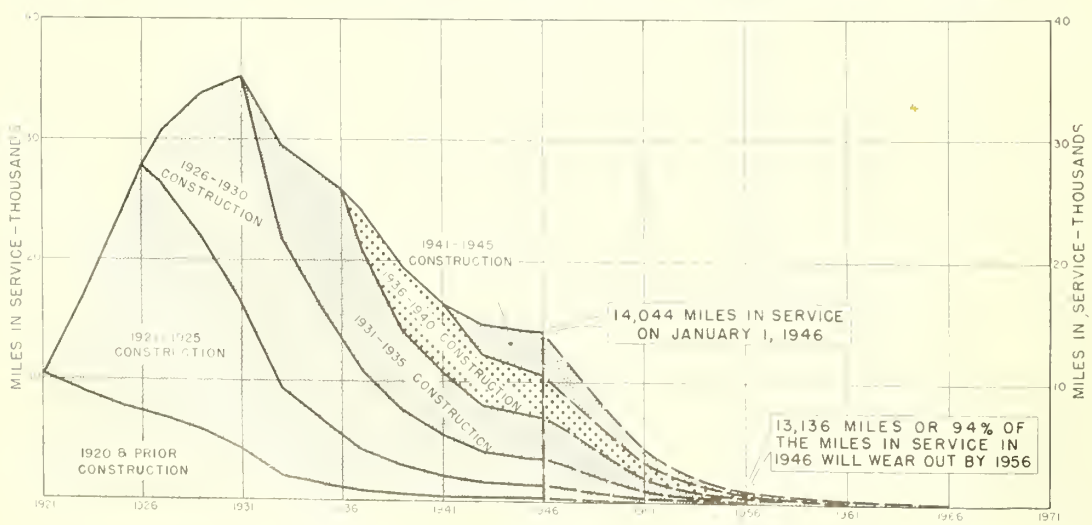


Figure 1.—Of the mileage of primary rural highways in service in 1946:

44 percent of the high-type surfaces will be worn out by 1956;


55,797 MILES IN SERVICE ON JANUARY 1, 1946
1941-1945 CONSTRUCTION
1936-1940 CONSTRUCTION
1931-1935 CONSTRUCTION
1926-1930 CONSTRUCTION
1925 & PRIOR CONSTRUCTION
41,264 MILES OR 74% OF THE MILES IN SERVICE IN 1946 WILL WEAR OUT BY 1956
MILES IN SERVICE-THOUSANDS

74 percent of the intermediate-type surfaces will be worn out by 1956;


14,044 MILES IN SERVICE ON JANUARY 1, 1946
1926-1930 CONSTRUCTION
1941-1945 CONSTRUCTION
1936-1940 CONSTRUCTION
1931-1935 CONSTRUCTION
1921-1925 CONSTRUCTION
1920 & PRIOR CONSTRUCTION
13,136 MILES OR 94% OF THE MILES IN SERVICE IN 1946 WILL WEAR OUT BY 1956
MILES IN SERVICE-THOUSANDS

94 percent of the low-type surfaces will be worn out by 1956.

increasing age and decreasing expectancy of the mileages in service.

LIFE OF INVESTMENT

A phase of the road-life work to which study is currently being given is the life of the dollar investment expended for the construction of highway facilities. This work is now under way in several States and is being conducted in such fashion that a continuous record is maintained of each year's construction investment and the amounts by which this investment is reduced at the time of resurfacing, reconstruction, or other method of retirement. Investment tabulations have now been prepared for Missouri, West Virginia, and Wisconsin.

Because of salvage at the time of retirement, the life of investment will exceed the service life of the surfaced mileage for all surface types. Indications are that the amount will vary anywhere from a few percent for some types having a low salvage at retirement to as much as several hundred percent in those instances involving stage construction or where there is extremely high salvage. During the period 1920–25, for example, approximately $9,500,000 was expended for the construction of 4,800 miles of gravel surfacing on the primary State highway system of Wisconsin. These gravel surfaces had an average life of 5.6 years, but the life of the dollar investment was 8.6 years or more than 50 percent greater than the life of the gravel as a surface. Unfortunately, the retirement experience to date has not been sufficient to warrant presentation in this report of similar estimates of the average life of dollar investment for all years and for each surface type.

It is possible, however, to show a comparison of the mileages constructed and retired with the corresponding investments which entered into their construction and which had been retired by January 1, 1941. From the comparison shown in table 11, it is apparent that the investment in various types is being retired at a slower rate than the mileage.

Since it is a common practice to apply bituminous mats to existing surfaces, the construction investments shown for intermediate-type roads do not reflect the total costs for a complete road including the base. To obtain the total cost for this type, it would be necessary to take into account the salvage value of gravel or stone and other type roads which may be utilized as a base for the intermediate-type road

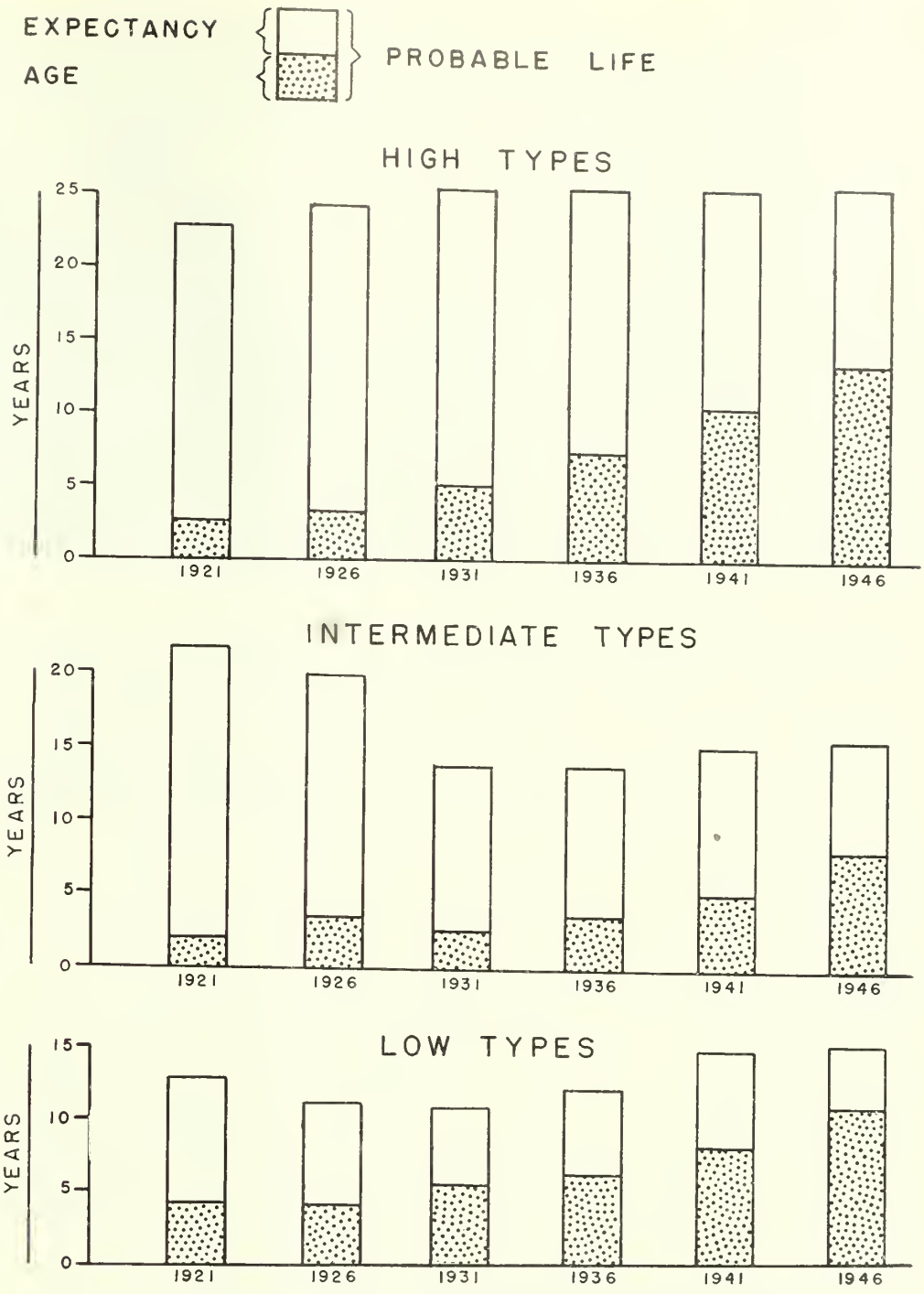


Figure 2.—Trends in average age, expectancy, and probable life, at 5-year intervals: 1921–46.

Table 11.—*Comparison of retirements of mileage with retirements of construction investment* [1]

Surface type as originally constructed	Total construction		Percentage retired up to Jan. 1, 1941	
	Mileage	Investment [2]	Mileage	Investment [3]
	Miles	*Dollars*	*Percent*	*Percent*
Low	21,766	53,300,776	89	57
Intermediate	29,108	[4] 56,304,360	68	26
High	12,489	265,681,512	19	12

[1] Compiled from data submitted by three States for rural State primary highways.
[2] These are actual expenditures for construction. No adjustment is made for the price index.
[3] These represent total losses of prior construction investments, no portion of which is used or usable in the road as of January 1, 1941. No depreciation is included in these percentages.
[4] In the construction of intermediate type, the construction dollars shown fall into two main categories:
(a) If the bituminous mat was placed over an existing low-type surface, the construction dollar investment for the intermediate type includes only the cost of the mat. In these cases there tends to be a high salvage for the low type when it is retired.
(b) If the base and surface mat were built at the same time as a single construction operation, the construction dollar investment for intermediate type includes the cost for both the mat and the base.

ROAD-LIFE STUDIES ESSENTIAL

The data presented in this report relate only to road surfaces. The road-life studies also embrace research in construction costs, maintenance costs, and salvage values for all elements of the highway, including grading

and structures. Investigations of these subjects will be extended as additional States bring their basic studies up to date and as these studies are continued and extended.

The objective of efficient and economical management of the highway program is to provide highway facilities at such locations and to such standards that they can absorb the inevitable and continuing changes in traffic requirements with the least effect upon the ability of the highway plant to provide maximum service at minimum cost. The data obtained from the road-life studies are among the essential facts needed to reach this objective.

APPENDIX

Surface Type Definitions

Soil-surfaced road.—A road of natural soil, the surface of which has been improved to provide more adequate traffic service by the addition of (1) a course of mixed soil having A–1 or A–2 characteristics, such as sand-clay, soft shale or topsoil, or (2) an admixture such as bituminous material, portland cement, calcium chloride, sodium chloride, or fine granular material (sand or similar material).

Gravel or stone road.—A road the surface of which consists of gravel, broken stone, slag, chert, caliche, iron ore, shale, chat, disintegrated rock or granite, or other similar fragmental material (coarser than sand) with or without sand-clay, bituminous, chemical, or portland-cement stabilizing admixture or light penetrations of oil or chemical to serve as a dust palliative.

Bituminous surface-treated road.—An earth road, a soil-surfaced road, or a gravel or stone road to which has been added by any process a bituminous surface course, with or without a seal coat, the total compacted thickness of which is less than 1 inch. Seal coats include those known as chip seals, drag seals, plant-mix seals, and rock-asphalt seals.

Mixed bituminous road.—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material, mixed with bituminous material under partial control as to grading and proportions.

Bituminous penetration road.—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material bound with bituminous material introduced by downward or upward penetration.

Bituminous concrete, sheet asphalt, or rock-asphalt road.—A road on which has been constructed a surface course 1 inch or more in compacted thickness consisting of bituminous concrete or sheet asphalt, prepared in accordance with precise specifications controlling gradation, proportions, and consistency of composition, or of rock asphalt. The surface course may consist of combinations of two or more layers, such as a bottom and a top course or a binder and a wearing course.

Portland-cement concrete road.—A road consisting of portland-cement concrete with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Brick [1] *or block road.*—A road consisting of paving brick, stone block, wood block, asphalt block, or other form of block, with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Methods of Retirement

Resurfacing.—Roads which are resurfaced or used as a base for the replacement type are so classified when the old surface is utilized more or less intact (with the exception of necessary scarifying, reshaping, or partial reworking of the surface) in the new construction which retires the old surface. Examples of this method are the retirement of a soil-surfaced road by surface treating, or the retirement of a gravel or stone road by utilizing it as a base or foundation for a mixed bituminous road or a bituminous penetration road. For surfaces which are retired by this method, it is obvious that the new or replacement construction must necessarily be along the same alinement and practically the same grade.

Reconstruction.—When surfaces are retired by reconstruction, there is little or no salvage of the old surface and base into the new type constructed. This classification includes old surfaces and bases that are torn up and not re-used. Usually, for types that are retired by this method, the replacement type is built along the same general alinement (generally within the limits of the existing right-of-way) involving only minor improvements in horizontal curvature. Substantial improvements are usually made with respect to grades, however.

Abandonment.—When the new construction is on new location, the old road is classified as abandoned when it is no longer maintained or kept in service at public expense. The abandoned road may revert to a private road, be barricaded to public travel, or torn up and removed. Sometimes, because of changes in land usage, such as abandonment of factories, and removal or construction of railroad facilities, roads may be abandoned without involving new construction that may be considered as replacing the mileage abandoned.

Transfer.—A retirement by transfer is similar to an abandonment except that the old road is continued in service after being dropped from the State or Federal-aid system by being maintained and resurfaced or reconstructed, when necessary, by the county or other authority responsible for the upkeep of the roads not on the State or Federal-aid system. A transfer is not a retirement in the sense that the road has rendered its total service to the public, but merely that it has rendered its complete service as a primary State or Federal-aid highway. Retirements by transfer are generally the result of functional obsolescence involving alinements and grades which are unsatisfactory for existing traffic conditions. A new road is built on new alinement and improved grades, and the old road remains in service usually because of the necessity of providing for local traffic usage. After the new road is placed in service on the State or Federal-aid highway system, the State will no longer desire to continue responsibility for further upkeep of the old road, and the county or other local authority generally takes over this responsibility. If the road is entirely discontinued from service it is considered an abandonment.

[1] Vitrified paving-brick roads were reported by the States separately from other types of brick or block roads. Because of the small mileages involved, these two types are combined in this report. Approximately 99 percent of the construction of these two types included in the report is vitrified paving brick.

Sheet-asphalt test sections on Connecticut Avenue, Washington, D. C.

Selection of Reference Densities for Bituminous Pavement Specifications

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported by H. M. REX, Materials Engineer

Few terms in asphalt paving technology are used so indiscriminately and are so liable to misinterpretation as the term "density". As applied to compacted bituminous mixtures, it is used to denote bulk specific gravity, unit weight, and degree of denseness or volume percentage of solids. Similarly, the terms "maximum density" and "theoretical maximum density" are very often used in specifications unaccompanied by sufficient definitive language; in such cases they are open to misunderstanding on the part of job engineers and contractors. Selection of a reasonable relative density value to designate the end-point of compaction during construction is likewise dependent on understanding the explicit meaning of the several expressions.

This paper discusses three different concepts of the expression "maximum density" and shows, by means of illustrative data, the degree to which reasonable relative density values for inclusion in a specification may vary with the respective concepts.

IN writing construction specifications for the various types of bituminous roadway surfaces, particularly those involving the hot plant-prepared mixtures, a relative density value is often used to delineate an end-point of compaction by the roller equipment. As usually written, such specifications require that rolling be continued until the bituminous surface has attained a specified percentage of a variously described maximum density. Sometimes the reference density is designated as that of a specimen, composed of the same mixture as used in the roadway, compacted in the laboratory. More often, however, the reference density is described merely as a "maximum density," or as a "theoretical maximum density."

The terms "maximum density" and "theoretical maximum density" are sometimes used synonymously. To many, however, the terms are considered as having quite different connotations. A "theoretical maximum density" is generally understood to be the calculated specific gravity of a voidless mass composed of bitumen and aggregate in the same relative proportions as used in the designed pavement mixture. By way of differentiation, the term "maximum density" is often used in referring to the observed specific gravity value of a laboratory-compacted specimen of the mixture, or a calculated specific gravity value based on the unit weight of the aggregate only, as determined by vibratory compaction in the laboratory. The numerical value obtained by any of these methods may be considered as being a "maximum density." When the value is derived (1), by calculating the specific gravity of an imaginary mass or (2), by using the unit weight of vibrated aggregate as the basis for computation, such a value could also come within the meaning of a "theoretical maximum density."

Which of the methods is employed in arriving at a reference specific gravity is less important than that the reference specific gravity be adequately identified in the specifications, and that the percentage of this reference specific gravity, or the relative density, to be required in the compacting process during construction be selected from a realistic viewpoint.

In most cases the use of a specific gravity value of laboratory-compacted specimens of the mixture as the reference maximum density will be found to be most satisfactory from a practical viewpoint. This procedure will, of course, entail a certain amount of work in ascertaining the compactive effort required with the use of laboratory equipment to produce the highest density for a given mixture without causing, at the same time, excessive crushing of the aggregate particles.

In laboratories equipped with suitable vibratory compaction equipment,[1] an equally

[1] *A new vibratory machine for determining the compactibility of aggregates*, by J. T. Pauls and J. F. Goode, PUBLIC ROADS, vol. 20, No. 3, May 1939.

Method of cutting cores for laboratory testing.

satisfactory method consists in consolidating to maximum density the uncoated aggregate only, and using the unit weight so obtained, with the specific gravity of the aggregate, in computing the reference maximum density value. The manner in which each of these laboratory procedures is used to develop a reference maximum density value will be fully explained later in this discussion.

RELATIVE DENSITY REQUIREMENT SHOULD BE PRACTICAL

When the calculated specific gravity of a theoretical voidless mass is used as the reference point for specification purposes, the relative density required for satisfactory compaction during construction should be selected with the greatest attention to practical considerations. If this is not done the limit may be so high as to produce some very undesirable effects in the pavement structure or, indeed, as to be impossible of attainment. For example, assume that an aggregate intended for use in bituminous concrete contains, in its densest possible arrangement of particles, 22 percent of voids, and that the optimum bitumen content appears to be 14 percent by volume. Assume also that the minimum relative density of the pavement, as compacted during construction, is stated by the specifications to be 95 percent of the specific gravity of a voidless mass. Manifestly this relative density could only be obtained, if it could be obtained at all, by crushing the aggregate to a distinctly undesirable degree.

Excellent illustrative information relating to this subject of density is provided by some incidental data collected in connection with a laboratory and field study of the alterations in a sheet-asphalt pavement resulting from age. This study was undertaken cooperatively by the District of Columbia Department of Highways and the Bureau of Public Roads, and was initiated in 1935–36. In all, 10 test sections of sheet-asphalt pavement were constructed on Connecticut Avenue in Washington, D. C., one of the city's most heavily traveled streets. The test sections were characterized by a number of mixture design variables. Compaction was not included as a variable, hence the construction procedures followed in all the sections were the same as those followed in usual sheet-asphalt contract work in that city. Cores, 2 inches in diameter, were cut from each of the sections shortly after construction and periodically thereafter over a 10-year period. Eighteen cores were cut from each section at each sampling. These cores were tested in the laboratory in a number of ways, the specific gravities of the top inch of each core being included in the test results. The illustrative values given in this discussion consist of the average of each set of 18 specimens cut from the 10 sections.

The design compositions, by weight, of the mixtures used in the 10 sections are given in the first part of table 1, together with the specific gravities of the asphalts. The apparent specific gravity of the combined mineral aggregate showed little variation within the range of weight proportions of sand and filler used. Many tests established a value of 2.674 for these combined materials.

COMPUTATION OF VOLUMETRIC COMPOSITION

Blends of sand and filler used in the mixtures were tested in the laboratory for vibratory density, and the unit weights of the vibrated aggregate are shown in the fifth column of table 1. Using these unit weights, together with the weight proportions and specific gravities of the asphalts, it is possible to calculate the volumetric composition, including air voids, of the mixtures theoretically compacted to their densest condition. These values are shown in the central part of table 1.

The following example will serve to illustrate the method used in computing these values. The unit weight of the aggregate used in section No. 1 was found, by vibratory test, to be 1.89 grams per cubic centimeter, or 189 grams per 100 cubic centimeters. The volume of the aggregate, since its specific gravity is 2.674, will be $189 \div 2.674 = 70.7$ cubic centimeters. The proportions by weight of aggregate and asphalt in this section were 89 and 11 percent, respectively, hence the total weight of 100 cubic centimeters of such a mixture would be $189 \div 0.89 = 212$ grams. Subtracting 189 from 212 grams, the weight of asphalt is 23 grams. As shown in the column of specific gravities in table 1, the weight of a cubic centimeter of asphalt is 1.030 grams, hence the volume of asphalt occupied in 100 cubic centimeters of this mixture is $23 \div 1.030 = 22.3$ cubic centimeters. Thus, in a volume of 100 cubic centimeters of

Table 1.—*Composition of mixtures, by weight and by volume*

Mix No.	Specific gravity [1] of asphalt 25/25°C.	Design composition, by weight		Composition by volume, based on unit weight of aggregate compacted by vibration				Composition by volume, based on unit weight of mixtures compacted in laboratory [2]				
		Asphalt	Aggregate	Unit weight of vibrated aggregate	Composition			Unit weight of compacted mixture	Unit weight of voidless mass [3]	Composition		
					Asphalt	Aggregate	Air voids			Asphalt	Aggregate	Air voids
		Percent	*Percent*	*Gm. per cm.*^{3}	*Percent*	*Percent*	*Percent*	*Gm. per cm.*^{3}	*Gm. per cm.*^{3}	*Percent*	*Percent*	*Percent*
1	1.030	11.0	89.0	1.89	22.3	70.7	7.0	2.12	2.275	22.6	70.5	6.9
2	1.032	11.0	89.0	1.91	23.2	71.4	5.4	2.15	2.276	22.9	71.5	5.6
3	1.026	11.0	89.0	1.89	22.4	70.7	6.9	2.11	2.273	22.6	70.2	7.2
4	1.032	10.5	89.5	1.86	21.3	69.6	9.1	2.08	2.291	21.2	69.6	9.2
5	1.028	11.0	89.0	1.89	22.4	70.7	6.9	2.12	2.274	22.7	70.6	6.7
6	1.027	11.0	89.0	1.90	22.4	71.0	6.6	2.13	2.273	22.8	70.9	6.3
7	1.030	10.5	89.5	1.89	21.4	70.7	7.9	2.10	2.290	21.4	70.3	8.3
8	1.024	10.5	89.5	1.90	21.5	71.0	7.5	2.09	2.287	21.4	70.0	8.6
9	1.022	11.0	89.0	1.91	23.5	71.4	5.1	2.10	2.270	22.6	69.9	7.5
10	1.022	10.5	89.5	1.87	21.5	69.9	8.6	2.11	2.286	21.7	70.6	7.7

[1] The specific gravity of the aggregate, in all mixes, was 2.674.
[2] Compaction using load of 3,000 pounds per square inch.
[3] Calculated.

this mixture, assuming the aggregate to be compacted to the density indicated in the vibratory test, there are, theoretically, 70.7 cubic centimeters of aggregate solids and 22.3 cubic centimeters of asphalt. There will be, then, $100-(70.7+22.3)=7.0$ cubic centimeters of air voids. Similar computations for the other nine sections indicate minimum air void contents ranging from 5.1 to 9.1 percent. In other words, the maximum relative densities of the sections, according to this type of concept, range from 90.9 to 94.9 percent of corresponding theoretical densities of a voidless mass.

Where equipment to make the vibratory test on the aggregate itself is not available, it is possible to use another method of arriving at a reasonable maximum density reference value. Cylindrical specimens of the mixture may be molded in the laboratory, using the same proportions of aggregate and asphalt as are to be used in actual construction. As a general rule, a static pressure of 3,000 pounds per square inch has been found to be a satisfactory molding load. Experimentation will indicate if deviation from this pressure is required for the particular materials that are to be used.

The application of this method in calculating maximum densities for the mixtures of the sheet-asphalt test sections is illustrated by the results shown in the last part of table 1. In this case cylinders, 2 inches in diameter and 1 inch in height, of each of the 10 design mixtures were molded while hot in the laboratory, using a pressure of 3,000 pounds per square inch. Specific gravity tests on the molded specimens established the unit weights for the 10 mixtures shown in the table.

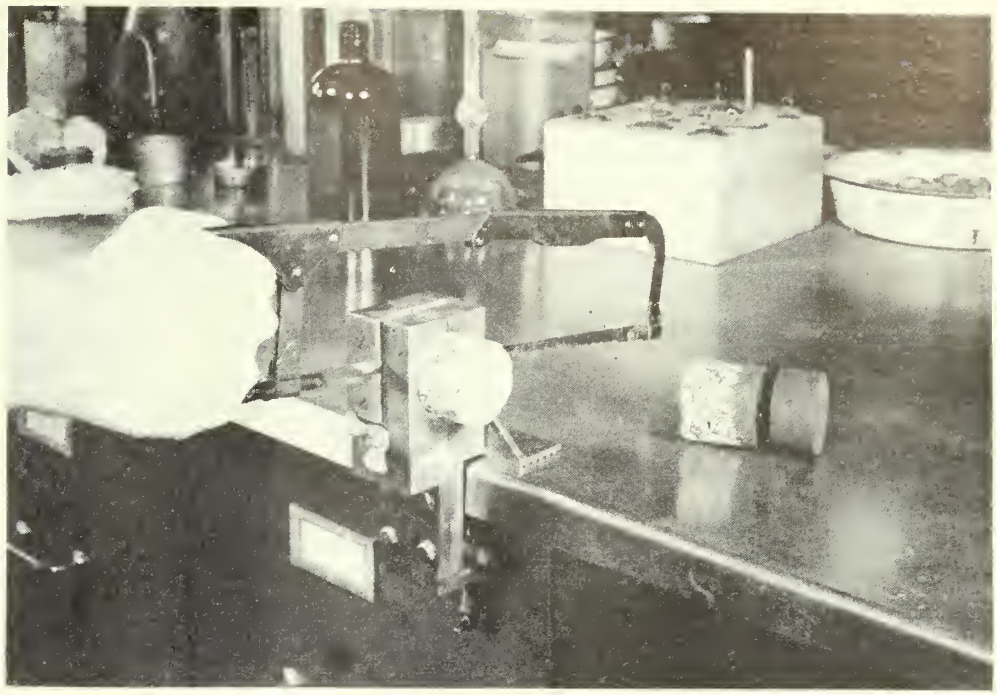
The other values were computed as follows:

$$U_v=\frac{100}{\frac{W}{G}+\frac{W_1}{G_1}}$$

$$A_v=\frac{UW_1}{G_1}$$

$$C_v=\frac{UW}{G}$$

$$E=100-(A_v+C_v)=100-\frac{100U}{U_v}$$



Sawing top inch of sheet-asphalt cores for density determination.

Where:

U_v = unit weight of a voidless mass (theoretical maximum density), gm. per cm³.
W = percentage of asphalt, by weight.
W_1 = percentage of combined filler and sand, by weight.
G = specific gravity of asphalt.
G_1 = apparent specific gravity of combined filler and sand.
A_v = percentage of aggregate, by volume, in the compacted mixture.
U = unit weight (bulk specific gravity) of the compacted mixture, gm. per cm³.
C_v = percentage of asphalt, by volume, in the compacted mixture.
E = percentage of air voids.

The highest possible densities for the 10 mixtures calculated by this method were found to range from 90.8 to 94.4 percent of voidless mass specific gravities. Although these values are slightly lower than those calculated on the basis of vibrated aggregate densities, the differences are small.

RELATION OF UNIT WEIGHTS TO RELATIVE DENSITIES

In table 2 are shown the values for theoretical maximum specific gravities or unit weights as computed on the three different bases, together with the relative densities of the top portions of cores from the pavement sections as sampled just after construction and again after 10 years of service. Only the section composed of mixture No. 2 attained a relative density of as high as 94 percent of a theoretical voidless mass, and that only after 10 years of service. On such a basis, all sections would have failed to pass specifications requiring a minimum relative density of 94 percent at the time of construction. It is likely that attempted enforcement of such requirements would have resulted in crushing of the aggregate particles or laminating and tearing of the pavement surface under the roller.

By contrast, all but one of the sections would have passed such a relative density requirement, if the maximum reference density were based on either compacted mixture specimens or aggregate compacted by vibration. Also, since maximum densities calculated on these two latter bases agree so closely with the observed ultimate densities of samples from the pavement, it would appear that specification requirements written around such maximum densities would be both logical and realistic.

Before stipulating field compaction density requirements, therefore, the specification writer should first of all decide upon a base or reference value with which the density of field samples taken during construction is to be compared. As has been stated above, either of the terms "maximum density" or "maximum theoretical density" may be used to denote the reference value. The value itself

Table 2.—*Densities of pavement samples cored at time of construction and 10 years later compared with maximum specific gravities calculated by different methods*

Mix No.	Maximum specific gravities			Relative densities of actual pavement samples					
	Based on vibratory test results	Based on laboratory compacted specimens	Based on assumption of voidless mass	Vibrated aggregate basis		Compacted mix basis		Voidless mass basis	
				Original	After 10 years	Original	After 10 years	Original	After 10 years
				Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	2.12	2.12	2.275	97.2	100.0	97.2	100.0	90.5	93.2
2	2.15	2.15	2.276	94.0	99.5	94.0	99.5	88.8	94.0
3	2.12	2.11	2.273	97.6	98.6	98.1	99.0	91.1	91.9
4	2.08	2.08	2.291	97.1	99.5	97.1	99.5	88.2	90.4
5	2.12	2.12	2.274	93.4	100.0	93.4	100.0	87.1	93.2
6	2.13	2.13	2.273	96.2	100.0	96.2	100.0	90.2	93.7
7	2.11	2.10	2.290	96.2	99.0	96.7	99.5	88.6	91.3
8	2.12	2.09	2.287	95.3	99.0	96.6	100.5	88.3	91.8
9	2.15	2.10	2.270	95.8	99.1	98.1	101.4	90.7	93.8
10	2.09	2.11	2.286	95.7	100.5	94.8	99.5	87.5	91.9
Average	------	------	------	95.8	99.5	96.2	99.9	89.1	92.5

may consist of (1) the calculated specific gravity of a theoretical voidless mass, (2) the observed specific gravity of a test specimen of the mixture compacted in the laboratory, or (3) a calculated specific gravity based on the unit weight of uncoated aggregate consolidated by vibration. In any case, the language used in the specifications should be explicit; as, for example, "samples taken from the pavement after compaction shall have a specific gravity not less than . . percent of the maximum density that has been determined for laboratory-compacted specimens of the mixture."

Selection of a minimum relative density value will depend upon the nature of the reference maximum density. In general, relative densities ranging from 95 to 98 percent of the maximum densities may be used if the latter are calculated on the basis of either laboratory-compacted mixtures or unit weights of aggregate vibrated in the laboratory. Considerably lower relative density values may be required if the calculated specific gravity of a theoretical voidless mass of the mixture is used as the reference maximum density. This will be particularly true in the case of surfaces composed of fine-grain mixtures, in some of which, as has been shown, it is possible to find air-void contents as high as 10 percent even after 10 years of service.

(Continued from page 188)

density, and moisture to a considerable depth is certain to exhibit area effects which differ from those found where such uniformity does not exist.

In the investigation being reported a special effort is being made to develop data on this question by testing the 5-foot embankment of uniform A-6 soil with load areas ranging in size from 1 to 7 feet in diameter. In addition to steel plates of 12, 18, 24, and 30 inches in diameter, concrete disks 42, 60, and 84 inches in diameter, as shown in figure 1, are being used as bearing surfaces. In these tests the incremental repetitional procedure is being employed.

In addition to the load-bearing tests there is under way a comprehensive study of temperature conditions in the pavement and subgrade. Sixteen thermocouples have been installed at selected locations and depths so that temperature gradients can be obtained in each pavement test section and in the subgrade beneath, to a maximum depth of 3 feet from the surface of the pavement. These temperatures are recorded at 1-minute intervals by an automatic instrument to an accuracy of $\pm 0.6°$ F. A view of the recorder is shown in figure 8. The locations of the thermocouples in the various pavement sections are shown in figure 2.

SUMMARY

As stated in the introduction, this report is presented primarily for the purpose of describing some innovations in plate-load testing techniques and testing equipment. The need for a rapid method of testing, a method which will permit the conduct of a number of tests in a relatively short period of time, has long been apparent. The method described, which gives, in addition to complete load-deflection relations, information pertaining to the behavior of the components of a flexible pavement structure, may serve partially to fulfill this need.

Highway Needs of the National Defense, a New Publication

Highway Needs of the National Defense, a report compiled by the Bureau of Public Roads in compliance with section 2 of the Federal-aid Highway Act of 1948, which directed the Commissioner of Public Roads to cooperate with the State highway departments in making a detailed study of the status of improvement and principal deficiencies of the National System of Interstate Highways, is being published as House Document 249, Eighty-first Congress, and will soon be available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C. The report is based on detailed data supplied by the State highway departments and embodies suggestions and recommendations of the Secretary of Defense.

The 40,000-mile interstate system is the trunk-line highway system of the United States, connecting most of the large cities of the country and serving the principal industrial centers. Including only 1 percent of the Nation's total road mileage, the system's rural sections serve 20 percent of the total traffic carried by all rural roads. Its urban sections, as thus far designated, serve more than 10 percent of the traffic moving on city streets. These main highways carry the bulk of the Nation's traffic and should be among the first considered for improvement. Large portions of the system are seriously obsolescent and are not suitable for the great number of vehicles attempting to use them.

The estimated cost of proposed improvements to correct existing deficiencies and to adapt the system to the needs of its present traffic is $11,266,000,000, based on 1948 construction costs. Of this sum, $5,293,000,000 would be expended on sections of the system in urban areas, and $5,973,000,000 would be assigned to projects on rural sections. As improvements are undertaken, the report recommends, ample provision should be made for the increased traffic volumes anticipated over a 20-year period. In this period, improvement must be undertaken on every mile of the system, requiring a capital investment of more than $500,000,000 a year. Completion of improvements on the system in a period far shorter than 20 years would result in much greater benefits.

The report advises the Congress to consider authorization of Federal appropriations earmarked for expenditure on the interstate system, apportioned among the States so as to permit substantially equal progress in correcting deficiencies in all States. In view of the interstate interest attaching to the system, the report recommends Federal participation in a ratio greater than the normal 50 percent of the cost. Federal authorizations for the Federal-aid primary, urban, and secondary systems should continue at rates not less than those established in the Federal-aid Highway Act of 1948, the report recommends; and the proposed authorizations for the interstate system should be in addition to these.

The report recommends that future highway legislation should provide for national emergencies in war or peace by authorizing diversion of highway appropriations to other purposes in time of need, such as construction of wartime-needed access roads and the emergency repair of roads damaged by floods and other major disasters. Road-building materials should be stock-piled as a precaution against possible need in the event of war.

Highway Capacity:

Practical Applications of Research

BY THE COMMITTEE ON HIGHWAY CAPACITY
DEPARTMENT OF TRAFFIC AND OPERATIONS
HIGHWAY RESEARCH BOARD

Reported by
O. K. NORMANN, Chairman,
Chief, Section of Traffic Operations,
Highway Transport Research Branch,
Bureau of Public Roads

and W. P. WALKER, Secretary,
Highway Engineer,
Highway Transport Research Branch,
Bureau of Public Roads

Because a large part of the basic research and analysis upon which this study is founded was conducted by the Bureau of Public Roads, the Highway Research Board considers it fitting that the report be published by the Bureau of Public Roads. Much of the basic research has, indeed, already been published in PUBLIC ROADS magazine. But the cooperative efforts of the Bureau of Public Roads, the Highway Research Board Committee on Highway Capacity, and a large number of State, county, and city highway and traffic engineers have resulted in the amassment, from the length and breadth of the country, of a far greater assemblage of field observations than would have been possible under the aegis of a single organization. Without such a body of basic information, this report would have lacked the authority it holds. The text is largely the work of O. K. Normann and W. P. Walker, of the Bureau of Public Roads. The Committee members contributed valuable assistance, both in the review of fundamental concepts and in the preparation of text and illustrations.

A rational and practical method for the determination of highway capacity is essential in the sound economic and functional design of new highways and in the adaptation to present or future needs of the many existing roads and streets which must continue in use for extended periods of time. Basically, the subject concerns the effectiveness of various facilities in the service of traffic, and involves the many elements of highway design, vehicle and driver performance, and traffic control.

Highway capacity has been the subject of continuing study over a long period of years, and the literature in this field is extensive. Practically all consideration of the subject in the past, however, has been handicapped by insufficient breadth of scope and by lack of any considerable volume of accurate data.

The cooperative efforts of the Bureau of Public Roads, the Highway Research Board Committee on Highway Capacity, and many State, county, and city engineers, intensively applied in many places and for a number of years, have resulted in a great mass of field observations. Complicated and ingenious instruments have made feasible the collection of data in larger volume and greater accuracy than was heretofore possible, and have permitted the mechanical recording of information that could not otherwise be obtained at all.

The study has encompassed not only the capacities of rural highways with uninterrupted flow, which have been fairly well established in previous publications, but has undertaken to determine the capacities of intersections at grade, weaving sections, grade separations, and ramps, and the relation of hourly to annual average traffic volumes. Such information has never before been available in comprehensive form, particularly for urban facilities.

With such a broad base of fundamental data, and with such painstaking analysis of them, this study will undoubtedly be recognized as one of the most thorough in highway research history.

From this vast grist of basic facts, so minutely sifted and examined, far more than a work report has been produced. This document is truly a practical guide by which the engineer, having determined the essential facts, can design a new highway or revamp an old one with assurance that the resulting actual capacity will be as calculated.

HIGHWAY RESEARCH BOARD COMMITTEE ON HIGHWAY CAPACITY

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Introduction

HIGHWAY CAPACITY has been the subject of careful and painstaking study for more than three decades. Many able research men have made valuable contributions to the subject, which has increased in importance from year to year at a rate that probably parallels the rapid increases in traffic volumes and congestion on our streets and highways. The comparatively recent general acceptance of the importance of highway transportation in our national economy, and especially of adequate transportation facilities in our cities, where heavy traffic volumes require the construction of costly roadways, has greatly increased the need for reliable street and highway capacity information.

The primary reason for constructing streets and highways is to serve traffic, whether the traffic is local and originates along the particular street or highway, or whether its origins and destinations are in other areas. Basically, a study of highway capacity is a study of the effectiveness of the various facilities in serving traffic and involves the many elements of street and highway design, vehicle and driver performance characteristics, and traffic control measures which directly influence the movement of vehicles.

To be of value in the sound economic and functional design of new roadways, or in adapting to present or future demands many of the existing roadways which must continue in use for long periods into the future, the capacity criteria must include measures of such factors as speed and the relative interference between vehicles in addition to the number of vehicles that can pass a point on a given roadway in a specified period of time. It is of little value to know the quantitative measure without knowing the quality of service provided.

Previous Studies

The need for an authoritative presentation of facts concerning roadway capacity is illustrated in figure 1 and table 1, which show the capacity of a single traffic lane as published in a number of articles. Each source is similarly identified in both the figure and the table.

Nearly all of the curves or equations were calculated from the formula:

$$C = \frac{5{,}280V}{S}$$

Where C = capacity of a single lane, in cars per hour.

V = speed, in miles per hour.

S = average distance in feet from center to center of vehicles.

In some cases the spacing between vehicles at various speeds was assumed to be a linear function of the speed, while in others the spacing was assumed to be a function of the square or some other exponent of the speed. In a few instances the spacing was based on very limited field observations and actual measurements. In general, however, spacings were arrived at by the use of such factors as driver reaction time, braking distances, and coefficients of friction.

Although some of these curves come remarkably close to showing the same relationship between speed and traffic volume as the curves presented in a following section of this report for specific roadway conditions, the wide range in vehicular volumes for any particular speed has had the effect of confusing rather than enlightening engineers who have made serious attempts to apply the results to practical problems.

Basis of This Report

One of the primary reasons that the earlier highway capacity analyses were not based on sufficient factual information was the lack of instruments to measure accurately and conveniently the vehicle speeds and the spacings between vehicles under normal operating conditions. The development of instrumentation for this purpose has progressed rapidly since 1934. Vital information pertaining to the actual driving practices of vehicle operators as related to other traffic on the highway and to the governing features of the highway itself can now be obtained for each individual driver, regardless of what the total traffic volume may be. Through the application of these means, certain laws or characteristics of traffic flow have been discovered or developed. This has made it possible to base this report on factual, technical data.

While compiling this report, the Highway Capacity Committee has reviewed all the available previously published information on the subject and has used the factual information obtained through investigations conducted by many individuals and organizations, especially the comprehensive traffic operation research investigations made by the Bureau of Public Roads in cooperation with the various State highway departments and other governmental agencies.

The Committee is especially indebted to the many city traffic engineers who contributed valuable data regarding intersection capacities, and to the Institute of Traffic Engineers for the help rendered in obtaining information on urban conditions.

The Committee is aware that highway capacities have been increasing over a period of years due to improved vehicle design, increased skill of drivers, and improved traffic control. With the introduction of four-wheel brakes, for example, closer spacing of vehicles was possible.

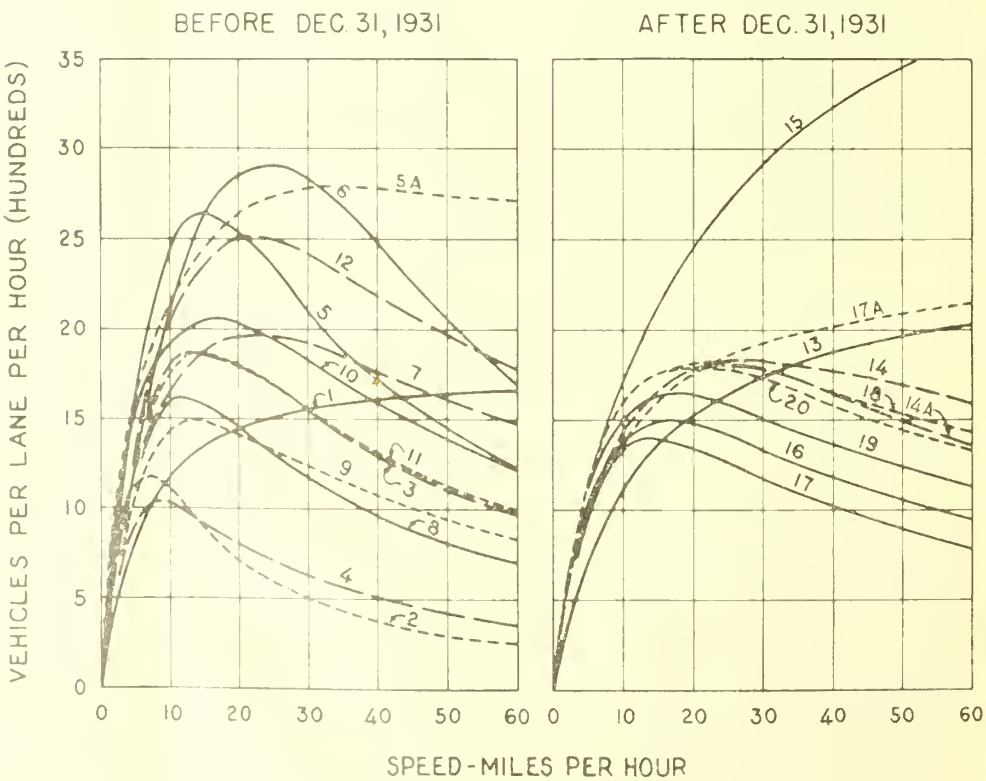


Figure 1.—Calculated capacities of a single traffic lane, from various sources.

Highway Capacity Stabilized

Improvements will continue to be made. The Committee believes, however, that capacities of highways are becoming stabilized insofar as the effects of the vehicle and the driver are concerned. Improvements which might tend to increase capacities will probably be offset, to some extent at least, by other factors. If the large number of rear-end collision accidents are to be appreciably reduced, for example, drivers must maintain somewhat longer distances between vehicles while operating in high traffic densities. A recent study of vehicle spacings under capacity conditions showed that 28 percent of the drivers could not have avoided a rear-end collision had the driver of the preceding vehicle suddenly applied his brakes. This assumes good brakes on both vehicles, and an extremely low value of 1 second for the combined perception and reaction time. The capacity figures in this report, therefore, are based on present-day vehicles and the manner in which they are currently operated on the various types of highway facilities.

Lake Shore Drive in Chicago has an outstanding record for moving huge volumes of traffic.

By correctly applying the various factors affecting street and highway capacities shown in this report, the engineer can evaluate quantitatively the deficiencies of long stretches of highways between important termini or between important intersections several miles apart. This will help to eliminate the common failure to recognize the need for improvement until a facility becomes badly congested. Instead, the need for improvement can be anticipated and the projects placed in their proper position in a long-range priority program for a street or highway system. Also, it should no longer be necessary to continually "chase rainbows" by increasing the capacity at one point only to find that the traffic bottleneck has been transferred to some other point along the route, with comparatively little overall benefit to traffic.

Table 1.—*Calculated capacities of a single traffic lane, from various sources*

Formula No.	Author	Year of publication	Safe following distance S when V is the speed in miles per hour	Assumed distance between cars at standstill (center to center)	Assumed reaction time	For maximum capacity—		Maximum capacity
						Speed	Spacing	
			Feet	*Feet*	*Seconds*	*M.p.h.*	*Feet*	*V.p.h.*
1	Schwanter	1924	$2.933V+14.7$	14.7	2.00	∞	(2.0 sec.)	1,800
2	French periodical	1924	$0.337V^2+14.7$	14.7	0	6.6	29.4	1,190
3	Lewis	1925	$0.0742V^2+0.733V+14.7$	14.7	0.50	14.1	39.7	1,870
4	Schaar	1925	$0.213V^2+1.47V+14.7$	14.7	1.00	8.3	41.6	1,050
5	Johnson	1926	$0.0667V^2+15$	15	0	15.0	30.0	2,640
5A	do	1928	$0.5V^{2.3}+15$	15	0	34.3	64.6	2,800
6	Kelker	1926	(1)	17.6		25.0	45.5	2,900
7	Highway Research	1927	$0.0366V^2+1.10V+17$	17	0.75	21.5	57.7	1,970
8	Weninger	1929	$0.109V^2+0.733V+14.7$	14.7	0.50	11.6	37.9	1,610
9	Ehlgotz	1929	$0.0773V^2+1.47V+14.7$	14.7	1.00	14.2	50.3	1,490
10	Daugherty	1930	$0.0556V^2+0.733V+15$	15	0.50	16.4	42.1	2,060
11	New York Regional	1931	$0.072V^2+0.733V+15$	15	0.50	14.4	40.6	1,870
12	Allan	1931	$0.0333V^2+0.733V+14$	14	0.50	20.5	43.0	2,520
13	Johannesson	1933	$2.20V+25$	25	1.50	∞	(1.5 sec.)	2,400
14	Massachusetts WPA	1934	$0.025V^2+1.47V+20$	20	1.00	28.3	81.5	1,830
14A	do	1934	$0.0102V^{2.3}+1.47V+20$	20	1.00	24.1	70.7	1,800
15	Greenshields	1935	$1.10V+21$	21	0.75	∞	(0.75 sec.)	4,800
16	Birula	1935	$0.0635V^2+1.47V+16.4$	16.4	1.00	15.8	55.5	1,500
17	Nevins	1939	$0.0833V^2+1.47V+16$	16	1.00	13.9	52.4	1,400
17A	do	1939	$2.20V+16$	16	1.50	∞	(1.50 sec.)	2,400
18	Clayton	1941	$0.0333V^2+1.47V+15$	15	1.00	21.2	61.1	1,830
19	do	1941	$0.05V^2+1.47V+15$	15	1.00	17.3	55.4	1,650
20	Gulstad	1941	$0.0373V^2+1.47V+15$	14.7	1.00	19.8	58.4	1,790

[1] Kelker, who did not list a formula of his own, assumed that at 10 m. p. h. the time spacing between vehicles is 0.5 second and at 60 m. p. h. is 2.0 seconds. These two points were joined with a straight line on semi-log cross-section paper.

Part I—Definitions

Road types: A freeway.

INTRODUCTION

The confusion that has existed concerning the meaning and shades of meaning of many terms used in traffic engineering practice has contributed, in some measure at least, to the wide differences of opinion regarding the capacity of various highway facilities. To cite but one of many examples, it is not uncommon to find the terms **high traffic density** and **high traffic volume** used synonymously or interchangeably. This practice is incorrect and creates misunderstanding in connection with highway capacities because **traffic volume** is a product of the traffic density and the traffic speed. As will be shown later, it is possible to have a very low traffic volume with a high traffic density. In fact, the highest traffic densities do occur when vehicles are practically at a standstill, in which case the traffic volume would approach zero.

The definitions given here are intended to be those most descriptive and most widely used in engineering practice. Most of them are based on current usage or are definitions already adopted by various organizations. There are, however, many cases in which a definition represents a combination of, or compromise between, definitions appearing in previously published material. The Committee's primary attempt has been to ascribe definite meanings to terms as they have been used in this report, thus minimizing likelihood of misinterpretation of its content.[1]

Included also are definitions for terms not used in this report, but the use of which will probably be necessary as the investigations of highway capacity are broadened to cover many of the specific conditions for which data are not now available.

[1] The definitions do not necessarily agree with those of other committees or associations, and may be subject to change to foster national uniformity of nomenclature.

Road types: An expressway in a rural area. Grade separation structures at minor cross roads such as the one shown here are seldom justified.

CAPACITY DEFINED

The term which is perhaps most widely misunderstood and improperly used in the field of highway capacity is the word **capacity** itself. The term **capacity,** without modification, is simply a generic expression pertaining to the ability of a roadway to accommodate traffic. Like the power of an engine, the capacity of a roadway must be rated by some standard before it can be expressed intelligibly. Just as the ability of any given pump to discharge a liquid is dependent upon such factors as the nature of the liquid, the speed of the pump, and the size of the discharge pipe, so the capacity of a roadway depends upon a number of conditions. Composition of traffic, roadway alinement, number and width of lanes, and vehicular speeds are a few of these conditions which may be referred to collectively as the **prevailing conditions.**

The prevailing conditions may be divided into two groups: (1) those that are determined by the physical features of the roadway; and (2) those that are dependent upon the traffic using the roadway. The first group, none of which change unless some construction or reconstruction work is performed, are referred to as the **prevailing roadway conditions.** The second group, any of which may change or be changed from hour to hour or during various periods of the day, are referred to as the **prevailing traffic conditions.**

Basic Capacity

There are three levels of roadway capacity that are of utmost importance in any discussion of the subject. The first of these the Committee has decided to call the **basic capacity. It is the maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained.**

Basic capacity for rural roads and urban expressways with uninterrupted flow is expressed in terms of passenger cars per lane or roadway per hour. Basic capacity for street intersections applies to the roadway width on one approach and is the rate of flow expressed in terms of passenger cars per hour, during the period that the flow is not interrupted. Two roadways having the same physical features, therefore, have the same basic capacity regardless of the prevailing traffic conditions.

Possible Capacity

The second level of capacity is **the maximum number of vehicles that can pass a given point on a lane or roadway during one hour, under the prevailing roadway and traffic conditions.** This level of capacity, which the Committee has decided to call the **possible capacity,** is a positive quantity. **It is the volume of traffic that cannot be exceeded in**

actuality without changing one or more of the conditions that prevail. It is necessary always to state the conditions under which an expressed possible capacity is applicable. For example, it would be incorrect to state that the possible capacity of a two-lane highway is 2,000 vehicles per hour. If, however, the statement be amplified to say that the possible capacity of a level, tangent, two-lane highway with a 24-foot surface, free from lateral obstructions within 6 feet of its edges, and with no major intersections at grade, is 2,000 passenger cars per hour, then it is substantially complete and correct.

Practical Capacity

The third level of capacity is **the maximum number of vehicles that can pass a given point on a roadway or in a designated lane during one hour without the traffic density being so great as to cause unreasonable delay, hazard, or restriction to the drivers' freedom to maneuver under the prevailing roadway and traffic conditions.** This type of capacity the Committee has decided to call **practical capacity.** Because the phrase **unreasonable delay or restriction to the drivers' freedom to maneuver** is somewhat subjective, the establishment of the volume at which practical capacity is reached depends in large measure upon individual judgment. The reader will find, however, that the body of the report develops scientific means of rationalizing the extent to which the driver of a vehicle is deprived of his freedom to maneuver at various traffic volumes by considering the amount that his speed and other factors are restricted by other vehicles on the highway. By using this as a criterion, the Committee has found it possible to recommend definite ranges of practical capacities for various roadway and traffic conditions.

It is the practical capacity which is of primary interest to those striving to provide adequate highway facilities. The design engineer will plan his improvements with an adequate practical capacity to meet anticipated volumes of traffic on the facility. To him, a more descriptive term might be **design capacity,** but the only difference is that the highway has a design capacity during the planning stage, and a practical capacity after it is constructed. One is objective, the other reality, but both may have the same numerical value. It seems illogical, therefore, to introduce a second term having essentially the same meaning.

Other Terms

Other terms, such as **satisfactory capacity, tolerable capacity,** and **intolerable capacity** have been variously used in previously published material to subdivide the range between practical and possible capacities. For urban conditions where traffic is regulated by traffic signals, the report will show that the difference between practical and possible capacities is so small that intermediate levels are not needed. In rural areas, however, and on urban facilities having expressway characteristics, there is indeed a wide range in volume between practical capacity and possible capacity. This may be thought of as a reservoir which can absorb an overload, with increasing inconvenience to drivers, after the practical capacity has been reached. The report will show that within this realm congestion increases in direct ratio with the traffic volume. Because the conditions that govern the degree of congestion which may be considered as tolerable are so local in character, the Committee chooses to refrain from any specific recommendations for these intermediate terms. Considered more important is the need to inform the reader of the effect of these intermediate traffic volumes. Having been thus informed, the local official will be better able to exercise sound judgment in deciding upon satisfactory or tolerable capacities for use in administering his available funds to the greatest advantage of the public.

These two photographs, taken at the same location within a short period of time, illustrate the difference between traffic volume and traffic density. The volume in both cases was about the same. The density, because of slow speeds, was far greater in the upper picture than in the lower picture, where speeds were appreciably higher. Heavy congestion often creates the false impression, in a photograph, of high volume. Actually, pictures of free-flowing, high-volume roads usually show only a few vehicles in the field of vision.

The three forms of capacity, **basic, possible,** and **practical,** are the ones used throughout the report, and they are used in the sense just described. Other terms and their definitions follow, not in alphabetical order, but grouped according to the subject to which they are most closely related. As an aid in locating the definition of any term listed, an index will be found immediately following the glossary.

I.—ROADWAY DEFINITIONS

Highway, street, or road. These are general terms denoting a public way for purposes of vehicular travel, including the entire area within the right-of-way. In rural areas, or in urban areas where there is comparatively little access and egress, a way between prominent termini is usually called a **highway** or a **road.** A way in an urban area with, or with provision made for curbs, sidewalks, and paved gutters, is ordinarily called a **street.**

Road types: A two-lane rural highway.

a. Control of access.—The condition where the right of owners or occupants of abutting land or other persons to access, light, air, or view in connection with a highway is fully or partially controlled by public authority.

(*1*) *Full control of access* means that the authority to control access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting crossings at grade or direct private driveway connections.

(*2*) *Partial control of access* means that the authority to control access is exercised to give preference to through traffic to a degree that, in addition to access connections with selected public roads, there may be some crossings at grade and some private driveway connections.

1. *Functional types:*

a. Arterial highway.—A general term denoting a highway primarily for through traffic, usually on a continuous route.

b. Expressway. **A divided arterial highway for through traffic with full or partial control of access and generally with grade separations at intersections.**

c. Freeway.—**An expressway with full control of access.**

d. Parkway.—**An arterial highway for noncommercial traffic, with full or partial control of access, and usually located within a park or a ribbon of parklike development.**

e. Major street or ***major highway.***—**An arterial highway with intersections at grade and direct access to abutting property, and on which geometric design and traffic control measures are used to expedite the safe movement of through traffic.**

f. Through street or *through highway.*—Every highway or portion thereof at the entrance to which vehicular traffic from intersecting highways is required by law to stop before entering or crossing the same and when stop signs are erected.

g. Local street or *local road.*—A street or road primarily for access to residence, business, or other abutting property.

2. *Cross-section components:*

a. Roadway.—That portion of a road which is improved, designed, or ordinarily intended for vehicular use. Divided roads, and roads with frontage roads, have more than one roadway. On undivided roads without frontage roadways, the roadway width normally lies between the regularly established curb lines or between the outer extremities of the shoulders, whichever is appropriate.

b. Frontage roadway.—**A roadway contiguous to and generally paralleling an expressway, freeway, parkway, or through street so designed as to intercept, collect, and distribute traffic desiring to cross, enter, or leave such facility and to furnish access to property which otherwise would be isolated as a result of controlled-access features. Sometimes referred to as a *service roadway*.**

c. Pavement.—That part of a roadway having a constructed surface for the facilitation of vehicular movement.

d. Shoulder.—That portion of a roadway between the outer edge of the paved surface and the curb or the inside edge

Road types: A three-lane highway.

of the ditch or gutter or original ground surface.

e. Curb.—A vertical or sloping member along the edge of a pavement or shoulder forming part of a gutter, strengthening or protecting the edge, and clearly defining the edge to vehicle operators. The surface of the curb facing the general direction of the pavement is called the "face."

(*1*) *Vertical curb.*—A curb whose face is a plane surface which is either vertical or inclined at an angle not exceeding 20 degrees with the vertical. Ordinarily it is not mountable, or is mountable with difficulty, by vehicles. Also called a *straight curb* or *normal curb.*

(*2*) *Sloped curb.*—A curb whose face is a plane surface which is inclined at an angle of at least 20 degrees, but not more than 60 degrees, with the vertical. It may be mountable or nonmountable by vehicles.

(*3*) *Lip curb.*—A curb whose face is a plane surface which is inclined at an angle of at least 60 degrees with the vertical and which is readily mountable by a vehicle.

(*4*) *Rolled curb.*—A curb, the face of which is S shaped in cross section, usually so constructed as to be mountable by a vehicle.

f. Separator.—An area or a device (other than a painted line or area) so located longitudinally between two roadways as to separate traffic flowing in the same or opposite directions and being so designed as to discourage or prevent passage by vehicles from the lanes on one side of the separator to those on the other.

(*1*) *Directional separator.*—A separator between traffic streams moving in opposite directions. If the directional separator is located between two roadways carrying through traffic in opposite directions, it is usually referred to as a ***Median.***

(*2*) *Lane separator.*—A separator between traffic streams moving in the same direction where the service rendered by the roadways on either side of the separator is essentially of the same character, as distinguished from that on a frontage roadway.

(*3*) *Outer separator.*—A separator between a frontage roadway and the roadway of a controlled-access highway or major street.

g. Speed change area.—**An added width of pavement adjacent to the through traffic lanes to enable vehicles entering a roadway to accelerate to a reasonable speed before merging with through traffic or to permit vehicles leaving the roadway to decelerate to the required speed after separation from through traffic has been accomplished.**

(*1*) ***Acceleration area.***—**A speed change area for the purpose of: (*a*) Enabling a vehicle entering a roadway to increase its speed to a rate at which it can more safely merge with through traffic; (*b*) Providing the necessary merging distance; and (*c*) Giving the main roadway traffic the necessary time and distance to make appropriate adjustments.**

(*2*) ***Deceleration area.***—**A speed change area for the purpose of enabling a vehicle that is to make an**

Road types: A one-way street.

exit turn from a roadway to slow to the safe speed on the curve ahead after it has left the main stream of faster-moving traffic.

3. Cross-sectional design:

a. Undivided road.—A road which has no directional separator, either natural or structural, separating traffic moving in opposite directions.

b. Divided road.—A two-way road on which traffic in one direction of travel is separated from that in the opposite direction by a directional separator. Such a road has two or more roadways.

4. Width in lanes:

a. Two-lane road.—An undivided two-way road having one lane for traffic in each of two opposing directions.

b. Three-lane road.—An undivided two-way road providing one lane for the exclusive use of traffic in each of two opposing directions and a third (center) lane for use by traffic in either direction in overtaking and passing.

c. Odd-lane road.—An undivided two-way road having an odd number of lanes, one of which may be used by traffic moving in either direction. A three-lane roadway is a specific type of odd-lane roadway.

d. Multilane road.—A road having two or more lanes for traffic in each direction, or four or more lanes for traffic in two directions. It may be one-way or two-way; divided or undivided.

5. Directional usage:

a. One-way road.—A road on which the movement of traffic is confined to one direction.

b. Two-way road.—A road on which traffic may move in opposing directions simultaneously. It may be either divided or undivided.

6. Traffic lane.—A strip of roadway intended to accommodate a single line of moving vehicles.

***a. Right lane or first lane.*—On any roadway, the lane on the extreme right, in the direction of traffic flow, available for moving traffic. Sometimes referred to as the outside lane on rural highways or the curb lane on city streets where parking is not permitted.**

***b. Left lane.*—(1) On a two-lane, two-way road, that lane which is to the left of the center line and which is normally used by traffic in the opposite direction.**

(2) On a multilane road, the extreme left-hand lane of those lanes available for traffic traveling in one direction.

***c. Center lane.*—On an undivided road having an odd number of lanes, the lane which may be used by traffic traveling in either direction.**

***d. Second lane, third lane, etc.*—On a multilane road, lanes to the left of the right lane (or first lane), available for traffic traveling in the same direction are designated "second lane," "third lane," etc., in that order.**

e. Parking lane.—A strip of roadway where vehicles may be legally parked but which otherwise would be available to moving traffic.

f. Left-turn lane.—A lane within the normal surfaced width, reserved for left-turning vehicles.

g. Right-turn lane.—A lane within the normal surfaced width, reserved for right-turning vehicles.

h. Separated turning lanes.—Added traffic lanes separated from the intersection area by an island or unpaved area.

Road types: An expressway having a directional separator (left) and a lane separator.

They may be wide enough for one-lane or two-lane operation.

i. Added turning lane.—A special lane for turning vehicles obtained by widening the normal roadway width at intersections.

j. Turn-out lane.—A special lane of limited length apart from the through traffic lanes provided for the use of vehicles making stops of short duration.

k. Car-track lane.—A lane occupied by streetcar tracks within the surfaced width of a roadway. Such a lane may or may not be available for use by free-wheeled vehicles.

7. Intersection.—The area embraced within the prolongation or connection of the lateral curb lines, or, if none, then the lateral boundary lines of the roadways of two highways which join one another at, or approximately at, right angles, or the area within which vehicles traveling upon different highways joining at any other angle may come in conflict. Where a highway includes two roadways (30) feet or more apart, then every crossing of each roadway of such divided highway by an intersecting highway shall be regarded as a separate intersection. In the event such intersecting highway also includes two roadways (30) feet or more apart, then every crossing of two roadways of such highways shall be regarded as a separate intersection.

Road types: A major street with left-turn lanes and separate signal indications for vehicles turning left.

Road types: A major street with frontage roads. Lamp posts are mounted in the outer separators.

a. Intersection leg.—That part of any one of the roadways radiating from an intersection which is outside of the area of the intersection proper.

(1) *Approach.*—That portion of a leg which is used by traffic approaching the intersection.

(2) *Exit.*—That portion of a leg which is used by traffic in leaving an intersection.

b. Three-way intersection.—A roadway intersection with three intersection legs. If one of these intersection legs is an approximate prolongation of the direction of approach of another, and if the third leg intersects this prolongation at an angle between 75 and 105 degrees, the three-way intersection is classed as a T intersection. If one leg is a prolongation of the approach of another, and the third leg intersects this prolongation at an angle less than 75 or greater than 105 degrees, it is classed as a Y intersection.

c. Four-way intersection.—A roadway junction with four intersection legs. If two of the intersection legs are approximate prolongations of the directions of approach of the other two, and the angle of intersection of these two prolongations is less than 75 degrees or more than 105 degrees, it is classed as a four-way oblique intersection. If two of the intersection legs are approximate prolongations of the other two legs, and the angle of intersection of these prolongations is 75 degrees or more, but not greater than 105 degrees, it is classed as a four-way right-angled intersection.

d. Multiway intersection.—A junction having five or more legs.

e. Rotary intersection.—A confluence of three or more intersection legs at which traffic merges into and emerges from a one-way roadway in a counterclockwise direction around a central area.

(7) *Traffic circle.*—A rotary intersection having a central area circular or oval in shape of sufficient size to produce weaving maneuvers in lieu of direct crossings between the various movements.

8. Highway grade separation.—A structure used to separate vertically two intersecting roadways, thus permitting traffic on the one road to cross traffic on the other road without interference.

a. Traffic interchange or *interchange.*—A system of interconnecting roadways in conjunction with a grade separation or grade separations providing for the interchange of traffic between two or more roadways or highways on different levels.

b. Access ramp or *ramp.*—An interconnecting roadway of a traffic interchange, or any connection between highway facilities of different levels, on which vehicles may enter or leave a designated roadway.

(1) *Inner loop.*—A ramp used by traffic destined for a left-turn movement from one of the through roadways to a second when such movement is accomplished by making a right-exit turn followed by a three-quarter-round right-turn maneuver and a right-entrance turn.

(2) *Outer connection.*—A ramp used by traffic destined for a right-turn movement from one of the through roadways separated by a structure to the second through roadway.

(3) *Direct connection.*—A form of ramp which does not deviate much from the intended direction of travel. The inner loop for left-turning movement is avoided by the use of separate structures. An outer connection is a direct connection for right-turning movements.

(4) *Exit turn.*—A turn at a ramp terminal where traffic leaves a through roadway and enters a ramp.

(5) *Entrance turn.*—A turn at a ramp terminal where traffic turns from a ramp and enters a through roadway.

Intersection types: A four-way intersection in a downtown area.

9. Weaving section.—A length of one-way roadway serving as an elongated intersection of two one-way roads crossing each other at an acute angle in such a manner that the interference between cross traffic is minimized through substitution of weaving for direct crossing of vehicle pathways.

***10. Sight distance.*—The length of roadway visible to the driver of a passenger vehicle at any given point on the roadway when the view is unobstructed by traffic.**

***a. Stopping sight distance.*—The distance required by a driver of a vehicle, traveling at a given speed, to bring his vehicle to a stop after an object on the roadway becomes visible.**

***b. Passing sight distance.*—The minimum sight distance that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably without interfering with the speed of an oncoming vehicle traveling at the design speed should it come into view after the overtaking maneuver is started.**

***c. Restricted stopping sight distance.*—A sight distance shorter than the stopping sight distance for the design speed.**

***d. Restricted passing sight distance.*—A sight distance shorter than the passing sight distance required to pass a vehicle traveling 10 miles per hour slower than the design speed.**

***e. Restrictive sight distance.*—A sight distance which, by reason of its inadequate length, causes a reduction in the operating speed and otherwise restrains the free movement of traffic under the prevailing conditions.**

II.—TRAFFIC CONTROL DEVICE DEFINITIONS

Traffic control devices.—A traffic control device is any sign, signal, marking, or device placed or erected for the purpose of regulating, warning, or guiding traffic.

1. Pavement markings:

a. Lane line.—A marked longitudinal line other than a center line separating two lanes.

b. Center line.—A marked center line (geometric medial line) of an undivided roadway.

c. Division line.—A line that designates the lanes available for traffic moving in each direction on an undivided roadway and that is to one side of the center line.

d. Barrier line.—A distinctive longitudinal pavement line which, when placed in proper relation to a center line, lane line, or division line, indicates that all traffic should keep to the right of such barrier line.

e. Insert.—A pavement marking accomplished by setting into or attaching to the pavement a material (other than paint) of contrasting color flush or practically flush with the surface.

f. Button.—A pavement marking consisting of an inflexible object attached to the pavement and projecting above the surface.

g. Word markings.—Word messages marked on the pavement to aid in the control of traffic.

h. Directional pavement markings.—Directional lines, arrows, or symbols marked on the pavement to aid in the control of traffic.

2. Traffic sign.—A device mounted on a fixed or portable support whereby official notice is given in the form of words or symbols,

Weaving sections: The distribution structure for the San Francisco-Oakland Bay Bridge (photo taken during construction in 1937).

for the purpose of regulating, warning, guiding, or informing traffic.

a. Stop sign.—A sign indicating that traffic shall stop before proceeding.

3. Traffic signal.—A signal, operated manually, electrically, or mechanically, by which traffic is alternately commanded to stop and permitted to proceed.

a. Interval.—The time during which the traffic indication of any particular traffic signal face does not change.

b. Cycle.—The total time required for one complete sequence of the intervals of a traffic signal.

c. Phase.—A part of the total time cycle allocated to any traffic movement receiving the right-of-way or to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals.

d. Through band.—The time in seconds elapsing between the passing of the first and the last possible vehicle in a group of vehicles moving in accordance with the designed speed of a progressive signal system.

e. Manual control.—Operation of a traffic signal by hand.

f. Automatic control.—Operation of a traffic signal by an electrical timing mechanism.

g. Traffic-actuated control.—A type of automatic control which is actuated, under specific conditions, by impulses from individual vehicles or pedestrians, or both.

h. Independent control.—Operation of a traffic signal installation not in coordination with any other signal.

i. Coordinated control.—Operation of two or more traffic control signal installations with definite interrelation.

j. Simultaneous system.—A signal system in which all signals always give the same indication to a given street at the same time.

k. Alternate system.—A signal system in which alternate signals, or groups of signals, give opposite indications to a given street at the same time.

l. Progressive system.—A signal system in which the various signal faces controlling a given street give GO indications in accordance with a timing schedule to permit (as nearly as possible) continuous operation of groups of vehicles along the street at a planned rate of speed, which may vary in different parts of the system.

m. Flexible progressive system.—A signal system in which the intervals at any signal may be independently adjusted to the traffic requirements at the intersection, and in which the GO indications at separate signals may be started independently at the instant which will result in maximum efficiency.

4. Safety zones and islands:

a. Safety zone.—An area or space set apart within a roadway for the exclusive use of pedestrians and which is protected or is so marked or indicated by adequate signs as to be plainly visible at all times.

Road Types: A depressed expressway.

When the area so set apart is for the protection of streetcar or bus passengers, it is known as a *loading zone.*

b. Pedestrian island.—A raised safety zone located in a cross walk.

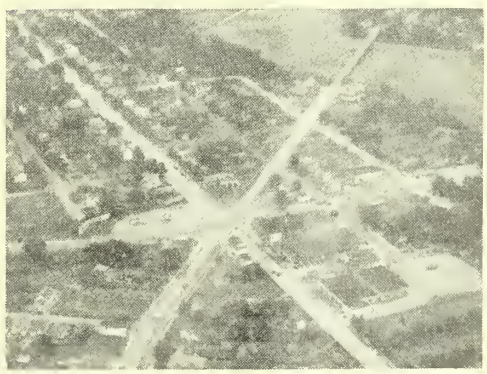
c. Loading island.—A raised safety zone especially provided at a regular streetcar stop, or at a bus stop when such is near the middle of the street, for the protection of passengers.

d. Traffic island.—Any restricted area permanently located in a roadway which provides structurally for the physical separation and sorting of traffic streams.

e. Channelizing island.—A traffic island located in a roadway area to confine specific movements of traffic to definite channels.

f. Divisional island.—A traffic island so located longitudinally in a roadway as to separate traffic streams flowing in the same or opposite directions. (*See also* "Separator").

g. Rotary island.—A traffic island located in the center of an intersection to compel movement in a counterclockwise direction and thus substitute weaving of traffic around the island instead of direct crossings of vehicle pathways.



Intersection types: A multiway intersection.

III.—TRAFFIC AND TRAFFIC OPERATIONS DEFINITIONS

A.—Composition of Traffic

1. Traffic.—All types of conveyances, together with their load, either singly or as a whole, while using any roadway for the purpose of transportation or travel.

a. Vehicle.—Any component of traffic. Unless otherwise qualified, the term vehicle will normally apply to free-wheeled vehicles as hereinafter defined.

b. Free-wheeled vehicle.—Any component of traffic not limited in its field of operation to rails or tracks.

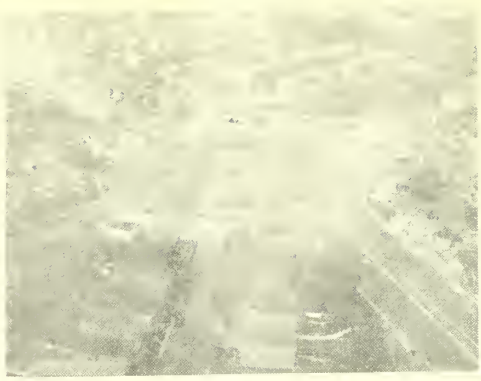
c. Passenger car.—A free-wheeled, self-propelled vehicle designed for the transportation of persons but limited in seating capacity to not more than seven passengers. It includes taxicabs, limousines, and station wagons, but does not include motorcycles.

d. Commercial vehicle.—A free-wheeled vehicle designed for the transportation of cargo other than passengers, and broadly referred to as a "truck." May be further classified as light-powered, medium-powered, and heavy-powered, and includes tractor-trucks, trailers, and semi-trailers when used in combination.

e. Motorbus (bus).—A free-wheeled vehicle having a self-contained source of motive power (as distinguished from a trolley coach), designed for the transportation of persons, and having a seating capacity of eight or more passengers.

f. Trolley coach or *trackless trolley.*—A vehicle, designed for the transportation of

Intersection types: A rural traffic circle.



Intersection types: An urban traffic circle.

passengers, which is propelled by electric power obtained from overhead trolley wires, but not operated upon rails.

g. Streetcar.—A single self-propelled vehicle restricted to operating on rails or tracks and designed for transporting passengers, principally within a municipality or contiguous area.

B.—Traffic Operation

1. Speed.—The rate of movement of traffic or of specified components of traffic, expressed in miles per hour.

a. Average speed.—The average of the speeds of all vehicles at a specified point on a given roadway, during a specified period of time.

b. Over-all speed.—The total distance traversed, divided by the total time required, including all traffic delays, expressed in miles per hour.

c. Average over-all speed.—The average of the over-all speeds of all vehicles on a given roadway during a specified period of time.

d. Optimum speed.—The average speed at which traffic must move when the volume is at a maximum on a given roadway. An average speed either appreciably higher or lower than the optimum will result in a reduction in volume. Also known as *critical speed.*

e. Design speed.—A speed selected for purposes of design and correlation of those features of a highway, such as curvature, superelevation, and sight distance, upon which the safe operation of vehicles is dependent. It is the highest continuous speed at which individual vehicles can travel with safety upon a highway when weather conditions are favorable, traffic density is low, and the design features of the highway are the governing conditions for safety.

f. Operating speed.—The highest over-all speed exclusive of stops at which a driver *can* travel on a given highway under prevailing conditions without at any time exceeding the design speed.

g. Speed difference.—The difference in the speeds of two successive vehicles moving in the same direction.

h. Average speed difference.—The average of the speed differences for all components of traffic on a given roadway during a specified period of time.

2. Delay.—The time consumed while traffic or a specified component of traffic is impeded in its movement by some element over which it has no control. Usually expressed in seconds per vehicle.

a. Fixed delay.—The delay to which vehicles are subjected during light traffic volumes or low densities. The delays experienced by a lone vehicle as a result of traffic signals or stop signs are fixed delays.

b. Operational delay.—The delay caused by the interference between components of traffic. The difference between travel times over a route during extremely low and high traffic volumes, and the time consumed while waiting at a stop sign for cross traffic to clear, are operational delays. Time losses resulting from congestion, from interference with parking vehicles, and from turning vehicles are also examples of operational delays.

Left-turn lanes: Jackson Boulevard at Lake Shore Drive, Chicago. Provision of three left-turn lanes eastbound, one right-turn lane eastbound, and two lanes westbound, reduces to a minimum the requisite time allotted to turning movements, thus affording greater time for through traffic on Lake Shore Drive.

3. Headway.—The interval of time between individual vehicles moving in the same lane measured from head to head as they pass a given point.

a. Average headway.—The average of the headways for vehicles on a given roadway during a specified period of time.

b. Minimum headway.—The existing headway when successive vehicles are moving as closely together as practical under prevailing conditions.

4. Stopping distance:

a. Vehicle stopping distance.—The distance traveled between the point at which the driver contacts the braking controls and the point at which the vehicle comes to rest.

b. Braking distance.—The total distance traversed by a vehicle while it is being brought to rest, measured from the position of the vehicle at the instant the brake shoe touches the brake drum.

c. Driver stopping distance.—The total distance traversed by a vehicle while it is being brought to rest, measured from the position of the vehicle at the instant the driver has an opportunity to perceive that he should stop his vehicle as quickly as possible. Includes the distance traveled during driver perception and reaction time.

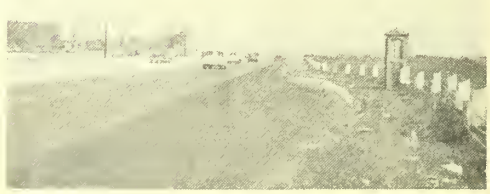
5. Weaving.—The act performed by a vehicle in moving obliquely from one lane to another, thus crossing the path of other vehicles moving in the same direction.

6. Merging.—The process by which drivers in two separate traffic streams moving in the same general direction combine or unite to form a single stream.

7. Volume.—The number of vehicles moving in a specified direction or directions on a given lane or roadway that pass a given point during a specified period of time, viz., hourly, daily, yearly, etc.

a. Average annual daily volume.—The total yearly volume divided by the number of days in the year; commonly abbreviated as ADT.

b. Maximum annual hourly volume.—The highest hourly volume that



Road types: A turn-out lane on a freeway (New Jersey Route 100). The telephone booth is well patronized.

occurs on a given roadway in a designated year.

c. Tenth highest annual hourly volume.—The hourly volume on a given roadway that is exceeded by only nine hourly volumes during a designated year.

d. Twentieth, thirtieth, etc., highest annual hourly volume.—The hourly volume on a given roadway that is exceeded by 19, 29, etc., hourly volumes during a designated year.

8. Density.—The number of vehicles occupying a unit length of the moving lanes of a roadway at a given instant. Usually expressed in vehicles per mile.

a. Average density.—The average number of vehicles per unit length of roadway over a specified period of time.

b. Critical density.—The density of traffic when the volume is at the possible capacity on a given roadway. At a density either greater or less than the critical density the volume of traffic will be decreased. Critical density occurs when all vehicles are moving at or about the optimum speed.

9. Variation in flow.—The fluctuation in traffic volume on a given street or highway with successive periods of the day, month, season, or year.

10. Traffic pattern.—A tabular or graphical representation of the fluctuation in traffic volume over a specified period of time. The volume during the increments of time used in the pattern may be expressed as numbers of vehicles or as the percentage which these numbers are of the average number for all the increments covered by the pattern. Different facilities may be said to have like traffic patterns if identical curves result when traffic volumes are shown as a percentage of the average volume for the period covered by the pattern.

a. Daily traffic pattern.—A traffic pattern wherein traffic volumes for each of 24 consecutive hours are shown. If the period of time is extended to include 168 consecutive hours it is termed a *daily traffic pattern for 1 week.* In like manner, daily traffic patterns for 1 month, 1 year, or any other period of time may be obtained.

Road types: Intersection on an expressway at grade having an added turning lane for left-turning vehicles. Separated turning lanes for right-turning vehicles are visible at the left of the picture.

Road types: Turn-out lanes on an expressway. These lanes might also be classed as a frontage road.

b. Weekly traffic pattern.—A traffic pattern wherein daily traffic volumes for each of 7 consecutive days are shown. If the period of time is extended to include 30 consecutive days the pattern is termed a *weekly traffic pattern for 1 month,* and if volumes for 365 consecutive days are shown it is termed a *weekly traffic pattern for 1 year.*

c. Seasonal traffic pattern.—A traffic pattern wherein the average daily traffic volumes for each of several consecutive months or seasons, as may be specified, are shown.

d. Yearly traffic pattern.—A tabular or graphical array of all the hourly traffic volumes in 1 year arranged in descending order of their magnitude, either as numerical values or as percentages of the traffic on the average day.

IV.—LAND USAGE AND DEVELOPMENT DEFINITIONS

1. Development:

a. Business development.—Land occupied predominantly by establishments used in the transaction of business.

b. Residential development. — Property which is in the main improved with residences.

c. Park development.—Land which is dedicated, maintained, and used for recreational purposes.

d. Agricultural development.—Cultivated land used for producing agricultural products.

2. Area or locality:

a. Urban area.—The area included within and adjacent to a municipality or other urban place of 5,000 or more in population.

b. Downtown area.—That portion of a municipality in and surrounding a business development where ordinarily there are large numbers of pedestrians and a heavy demand for parking space during periods of peak traffic or a sustained high pedestrian volume and a continuously heavy demand for parking space during business and industrial employment hours. This definition applies to industrial and business areas outside of, as well as those within, the central part of a municipality.

c. Intermediate area.—That portion of a municipality which is outside of a downtown area but generally within the zone of influence of a business development, characterized by moderately heavy pedestrian traffic and a somewhat lower parking turn-over than is found in a downtown area.

d. Outlying area.—A residential development, or a mixture of residential and commercial establishments, within the outer fringe of a metropolitan area, characterized by few pedestrians and a low parking demand or turn-over.

e. Rural area.—Any area not within a town, city, or metropolitan area.

Road types: An elevated multilane highway.

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[2] Columns are indicated as *a*, left-hand column; *b*, center column; and *c*, right-hand column.

Part II.—Maximum Observed Traffic Volumes

Introduction

One method for determining the load-carrying ability of a machine or material is to apply stresses in successive increments of known quantity until the item being investigated is no longer able to support the burden imposed on it. In the industrial laboratory the strength, or capacity, of a test sample is usually determined by loading it to destruction, the greatest load it will support being its capacity. The highway engineer has at his disposal a comparable means for measuring the ability of a roadway to accommodate traffic. His laboratory extends throughout the country and his test samples are the many sections of highway that are repeatedly exposed to the maximum load they can possibly bear. His tools are a simple traffic counter and a timepiece. Because this is perhaps the earliest applied of all methods for studying highway capacity, it is fitting that some of the results obtained by this means be introduced early in a discussion of the subject.

The analogy between the laboratory and the highway fails in one very important respect: In the laboratory the conditions under which the test is made can be carefully controlled, whereas the traffic engineer cannot readily alter either the roadway or traffic conditions that are prevalent during his test. He must note these conditions and make compensatory adjustments for the deviations from "standard" if his results are to be of any value. Because of failure to do this or to make similar adjustments in later applications of the data, the results obtained by this method have met with varying degrees of success.

Arlington Memorial Bridge, Washington, D. C. Low-speed approaches, lanes less than 12 feet wide, and high curbs limit the possible capacity of this roadway to about 4,000 vehicles per hour in one direction.

Table 2.—*Maximum observed hourly volumes on two-lane roads*

Route and location	Average lane width	Vehicles per lane per hour	
		In one lane	Average for all lanes
	Feet		
Colorado St. Bridge, Pasadena, Calif	14.0	1,710	1,215
Sumner Tunnel, Boston, Mass	10.8	----------	1,200
Posey Tube, Alameda, Calif	11.5	1,400	1,350
U S 3, Manchester, N. H	10.0	----------	944
U S 45, Libertyville, Ill. (31 m.p.h.)	9.0	1,510	904
Lincoln Tunnel (1940), New York	10.8	1,152	----------
U S 1, Old Lyme, Conn	11.5	1,135	887
U S 27, north of Jct. Route 21, Mich	10.0	----------	850
U S 6, West Hartford, Conn	10.0	----------	830
State Route 17, northwest of Goshen, N. Y	10.0	----------	801
Shirley Highway (October 1944), at Glebe Road, Va. (27 m.p.h.)	12.0	1,251	752
Baltimore-Washington Blvd., Maryland	10.0	----------	751
State Route 9, Middletown, Conn	10.0	1,127	714
L & N Bridge, Cincinnati, Ohio	7.0	988	575
U S 285, near State Route 75, Denver, Colo	9.0	836	----------
State Route 87, Fayetteville, N. C	11.0	750	----------

Table 3.—*Maximum observed hourly volumes on three-lane roads*

Route and location	Average lane width	Vehicles per hour	
		In one direction	Average for all lanes
	Feet		
State Route 35, Laurence Harbor, N. J	10.0	2,250	1,021
State Route 27, Rahway, N. J	10.0	1,905	761
State Route 27, West Scott Ave., Rahway, N. J	10.0	1,814	724
U S 1, Danvers, Mass	10.0	1,526	664
State Route 29, Union County, N. J	10.0	1,511	757
State Route 42, Camden County, N. J	10.0	1,391	664
U S 40, Richmond, Calif	10.0	----------	653
State Route 17, Carlstadt, N. J	10.0	1,197	633
U S 101, Oceanside, Calif	10.0	----------	632
State Route 3, Quincy, Mass	10.0	920	568

Two- and Three-Lane Roads

Tables 2, 3, and 4 show the maximum hourly volumes that have been observed on a number of important highway facilities when the traffic flow was continuous. In each table the maximum traffic volume for one lane and for the average lane is shown. Values under the heading "In one lane" represent the maximum hourly volumes recorded for a single lane, except that for three-lane roads they represent the maximum hourly volumes for one direction of travel. The "Average for all lanes" is for both directions of travel on a two- or three-lane roadway and for all lanes in one direction of travel on a multilane roadway. The highest observed volume for one lane may or may not have occurred at the same time as the highest observed volume for the average lane.

The highest volume recorded for a two-lane roadway is 1,350 vehicles per lane in the Posey Tube at Alameda, Calif. A maximum volume of 1,710 vehicles in one lane has been observed on the Colorado Street Bridge in Pasadena, Calif. The roadway width of the latter is 28 feet and therefore might have been used, to some extent at least, as three lanes. Several other two-lane roadways have accommodated volumes exceeding 1,000 vehicles per hour in one direction but, as far as is known, these two facilities and the Sumner Tunnel are the only two-lane facilities that have accommodated an average of more than 1,000 vehicles per hour per lane or a total volume exceeding 2,000 vehicles per hour.

Lake Shore Drive, Chicago, operating with six lanes in the direction of heavier traffic movement. Separators between pairs of lanes can be raised or lowered at will, thus affording great flexibility in meeting traffic needs during various periods of the day.

Particular attention is called to the volume for the Baltimore-Washington Boulevard as shown in table 2. Several articles on highway capacity incorrectly report that 1,502 vehicles per lane were observed in 1 hour on this facility when it was a two-lane road. Examination of the original material reveals, however, that this figure was the total for both lanes and not for each lane. Similarly, many articles show the capacity of the Holland Tunnel as 2,000 vehicles per lane per hour. Actually, the Holland Tunnel has never carried more than a total of 2,505 vehicles in two lanes, or an average of 1,253 vehicles per lane per hour, as shown in table 4. Since each tube of the Holland Tunnel is a two-lane, one-directional facility, it is classed as a four-lane road.

A special attempt was made to obtain counts showing high hourly volumes on three-lane roads. Only one example was found, however, of a three-lane road with a peak hourly volume exceeding 3,000 vehicles, or 1,000 vehicles per lane per hour. On many three-lane facilities, average annual volumes in excess of 12,000 vehicles per day have been recorded. In one case the annual average 24-hour volume was 19,040 vehicles per day. Even with these high daily volumes, however, the maximum hourly volumes shown in table 3 have not been exceeded.

Multilane Facilities

Only two multilane facilities, U S 1 near the airport at Newark, N. J., and Grand Central Parkway, New York City, both with four-lane divided roads, have been reported as accommodating in excess of 2,000 vehicles per lane per hour (table 4).

The Outer Drive in Chicago is one of the best illustrations of high lane capacities on a road with a total of more than four lanes. It carries in the neighborhood of 1,500 vehicles per lane per hour nearly every day during the afternoon rush period. Maximums of 1,958 vehicles per lane when operating with four lanes in each direction and 1,640 vehicles per lane when operating with six lanes in one direction have been recorded.

Maximums Cited Are Exceptional

The volumes shown in tables 2, 3, and 4 are, in most cases, the possible capacities of those particular facilities. They are maximums that have been recorded only once, although the traffic demand has probably been sufficient to reach or exceed these same volumes on many occasions. Compared with a laboratory test of a number of identical samples of steel, these traffic counts correspond to the one test sample that would yield under a heavier load than all the others. Traffic on Memorial Bridge in Washington, D. C., for example, has been counted innumerable times when the bridge was unable to accommodate the approaching volume of traffic, thus causing traffic to back up on the approaches as a result of congestion on or near the bridge. During hundreds of such observations this bridge carried no more than 3,600 vehicles per hour in one direction, or an average of 1,200 vehicles per lane. On one occasion, however, and only on one occasion as far as is known, this bridge carried a total of 4,227 vehicles in one direction, or an average of 1,409 vehicles per lane.

Furthermore, the data included in tables 2, 3, and 4 were selected as the highest group of hourly volumes from a much larger list showing maximum observed volumes for facilities that, in the judgment of the observer, were loaded to their capacity. It would therefore be technically unsound to assume that other facilities with a comparable number of lanes would have the same capacities as the highest or even the average of these few isolated examples.

The maximum observed volumes are shown primarily to acquaint the reader with the peak traffic that has been carried on some of the more heavily traveled routes throughout the country. They are also intended to illustrate the wide range in capacities of facilities seemingly alike in type but actually having wide differences in their physical as well as traffic characteristics. The reasons for these wide differences in capacity will become more apparent as the subject is developed in succeeding chapters.

Table 4.—*Maximum observed hourly volumes in one direction on multilane roads*

Route and location	Average lane width (feet)	Vehicles per lane per hour	
		In one lane	Average for all lanes
FOUR-LANE ROADS			
U S 1, Newark Airport, N. J.	12.4		2,275
Grand Central Parkway, west of Parsons Blvd., N. Y.	11.0	2,275	2,194
U S 73 and 75, Omaha, Nebr. (34 m.p.h.)	11.0		1,763
Cross Bay Blvd., Queens, N. Y.	8.0		1,566
Express Highway, St. Louis, Mo.		1,956	1,543
Fourteenth St. Bridge, Washington, D. C. (27 m.p.h.)	10.0	1,620	1,508
Queensboro Bridge, upper roadway, New York, N. Y.	11.25	1,861	1,488
Sunrise Highway, at Cross Bay Blvd., N. Y.	10.0		1,350
Outer Drive, Chicago, Ill.	11.25	1,395	1,349
Southern State Parkway, Troopers Lodge, N. Y.	10.0		1,264
Holland Tunnel, New York, N. Y. (28 m.p.h.)	10.0	1,404	1,253
Edison Bridge, South Amboy, N. J.	10.7		1,246
Manhattan Bridge, New York, N. Y.:			
Upper roadway	11.25	1,884	1,226
Lower roadway	8.75		912
Merritt Parkway, Greenwich, Conn.	13.0		1,158
Northern State Parkway, Troopers Lodge, N. Y.	10.0		1,136
Broadway Bridge, Portland, Oreg.	10.0		1,088
Williamsburg Bridge, New York, N. Y.	10.0		1,061
U S 20, Oxford, Mass.	10.0		1,042
FIVE-LANE ROADS			
Superior Bridge, Cleveland, Ohio	10.0		1,557
Queensboro Bridge, New York, N. Y.:			
Lower roadway, two lanes one-way	10.2	1,720	1,292
Lower roadway, three lanes one-way	10.2		1,195
SIX-LANE ROADS			
Henry Hudson Parkway, New York, N. Y.	11.0		1,666
Oakland Bay Bridge, San Francisco, Calif.	9.7		1,538
Leif Eriksen Drive, Chicago, Ill.	11.6		1,435
Memorial Bridge (30 m.p.h.) Washington, D. C.	10.0		1,409
Arroyo Seco Parkway, Los Angeles, Calif.	10.7		1,367
George Washington Bridge, New York, N. Y.	8.0		1,320
Delaware River Bridge, Camden, N. J. (four lanes one-way)	9.4		1,285
Superior Bridge (12.5 m.p.h.), Cleveland, Ohio	10.0	1,530	1,241
Michigan Ave., Chicago, Ill.	12.5		1,118
Carnegie Ave., Cleveland, Ohio	9.5	1,098	938
Aurora Bridge, Seattle, Wash.	9.3	1,111	903
Burnside Bridge, Portland, Oreg.	10.0		768
U S 1, Saugus, Mass.	11.3		746
Davison Expressway, Detroit, Mich.	11.0	1,013	678
EIGHT-LANE ROADS			
Outer Drive (37 m.p.h.), Chicago, Ill.	12.1	2,155	1,958
Curb lane (32 m.p.h.)	12.1		1,519
Second lane (35 m.p.h.)	12.3		2,155
Third lane (40 m.p.h.)	12.5	[1] 2,174	2,081
Fourth lane (40 m.p.h.)	11.7		2,077
Six lanes one-way	11.25		1,640
Figueroa Freeway, Los Angeles, Calif.	10.0		1,463

[1] The maximum in this lane occurred when the average lane carried only 1,681 vehicles per hour.

Part III.—Fundamentals of Highway Capacity

Introduction

The subject of street and highway capacity is not readily understood without a knowledge of the various factors which, either singly or in combination, affect the movement of vehicles. Principles of physics, dynamics, hydraulics, and the laws of various sciences have been applied from time to time to the movement of traffic with varying degrees of success. Although some of these laws are pertinent to traffic flow, a thorough knowledge of traffic movement has required the development of new principles.

Few drivers, if any, operate their vehicles in identically the same manner or react exactly the same under similar conditions. It is impossible, therefore, to predict the effect of various roadway and traffic conditions on an individual driver. It has been found, however, that the combined effect on traffic as a whole can be predicted with reasonable accuracy. This chapter is devoted to a few of the principles of traffic flow which relate to highway capacity.

Vehicle Spacings at Various Speeds

All drivers do not maintain the same distance spacing to the vehicle ahead when trailing at a given speed. In fact, the same driver in congested traffic will follow at different distances depending on the surroundings and his ability to match the speed of the preceding vehicle. Figure 2 shows the minimum distance spacings allowed by the average driver at different speeds for a few conditions. Similar curves based on thousands of observations for other conditions could be presented. The curves in figure 2 are, however, sufficient to show that the average driver increases the distance spacing between vehicles as his speed increases and that the spacing is also influenced by the characteristics of the highway.

Relation Between Speed and Basic Roadway Capacity

Using the data shown in figure 2, it is possible to determine the maximum number of passenger cars, one behind the other, that can pass a point in 1 hour at any given speed if this given speed is maintained by all vehicles. This is shown by figure 3. These curves have numerous practical applications if used correctly. **Under actual operating conditions it is possible to attain the traffic volumes shown for two-lane roads in one of the two lanes, but only upon the condition that there is no oncoming traffic in the other of the two lanes. Where traffic is moving in both directions on a two-lane road the combined volume in the two lanes may equal the values shown in figure 3. In fact, the total traffic volume over a section of two-lane, two-way road more than a few hundred feet long cannot exceed 2,000 vehicles per hour regardless of the distribution of traffic by directions. Likewise, the traffic volumes shown for three-lane roads can be attained in only two of the three lanes. In the case of four-lane roads, however, it is possible to attain the volumes as shown by figure 3 in each of the traffic lanes during the same time period.**

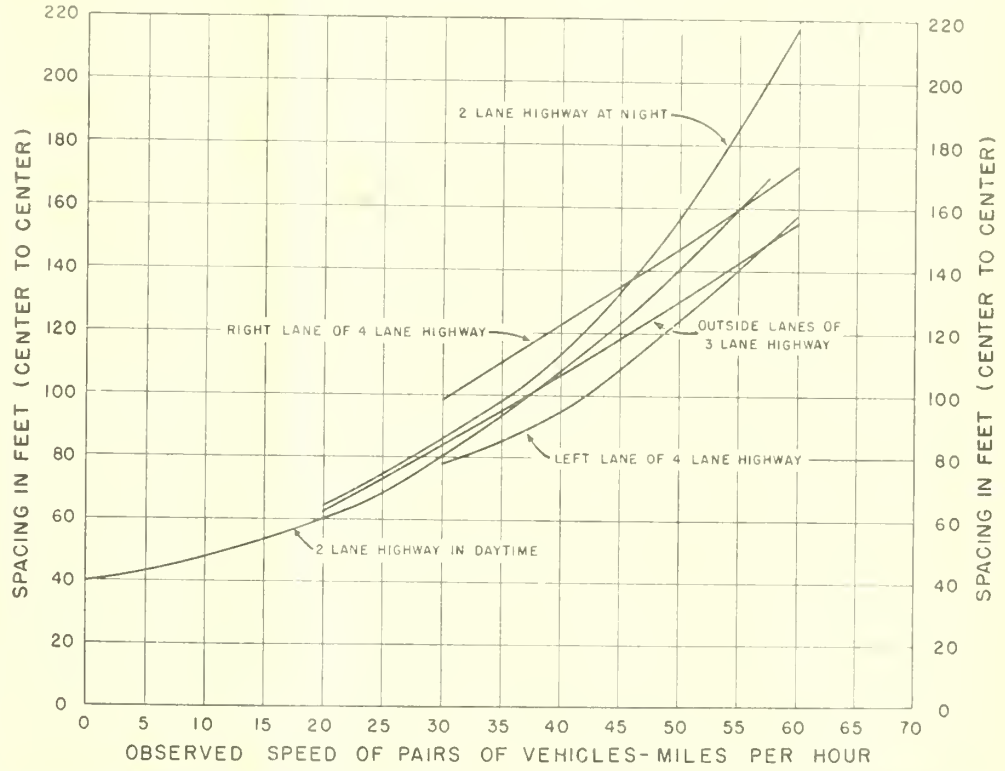

SPACING IN FEET (CENTER TO CENTER)
OBSERVED SPEED OF PAIRS OF VEHICLES-MILES PER HOUR
2 LANE HIGHWAY AT NIGHT
RIGHT LANE OF 4 LANE HIGHWAY
OUTSIDE LANES OF 3 LANE HIGHWAY
LEFT LANE OF 4 LANE HIGHWAY
2 LANE HIGHWAY IN DAYTIME

Figure 2.—Minimum spacings allowed by the average driver when trailing another vehicle, at various speeds.

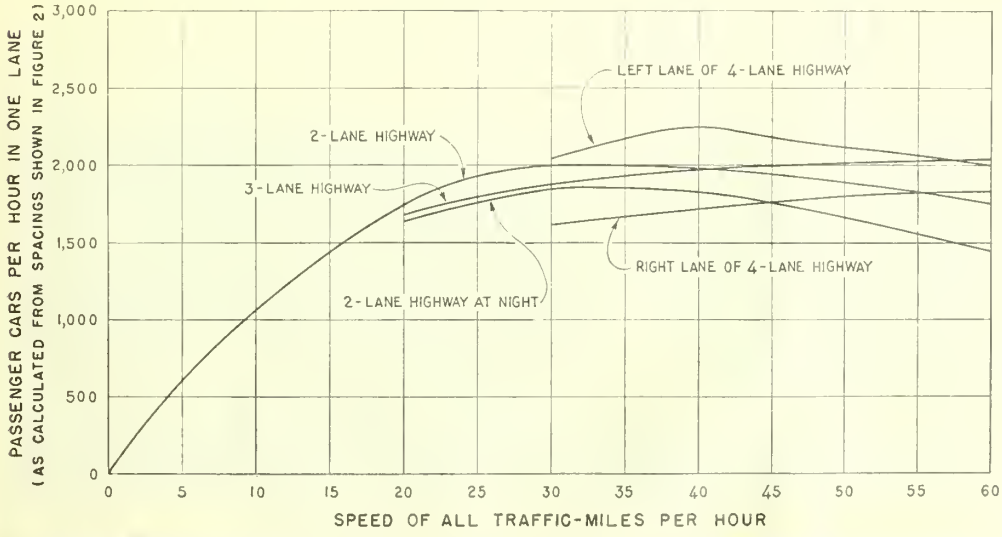

PASSENGER CARS PER HOUR IN ONE LANE (AS CALCULATED FROM SPACINGS SHOWN IN FIGURE 2)
SPEED OF ALL TRAFFIC-MILES PER HOUR
LEFT LANE OF 4-LANE HIGHWAY
2-LANE HIGHWAY
3-LANE HIGHWAY
RIGHT LANE OF 4-LANE HIGHWAY
2-LANE HIGHWAY AT NIGHT

Figure 3.—Maximum capacity of a traffic lane, based on average spacings between pairs of vehicles traveling at the same speed.

The relation between average speed and traffic volume as shown by figure 3 holds true in actual practice. **The highest volumes per lane occur on roads where vehicles travel between 30 and 40 miles per hour at the time the roads are accommodating their maximum possible capacities.** For example, a two-lane road that handles its highest possible capacity when the vehicles are traveling 15 miles per hour will have only three-fourths the capacity of a road that handles its possible capacity at 30 miles per hour. Likewise, the road that handles its possible capacity at 30 miles per hour will only be able to handle half as much traffic whenever some abnormal condition causes the drivers to reduce their speeds to 9 miles per hour. Any traffic variable or any

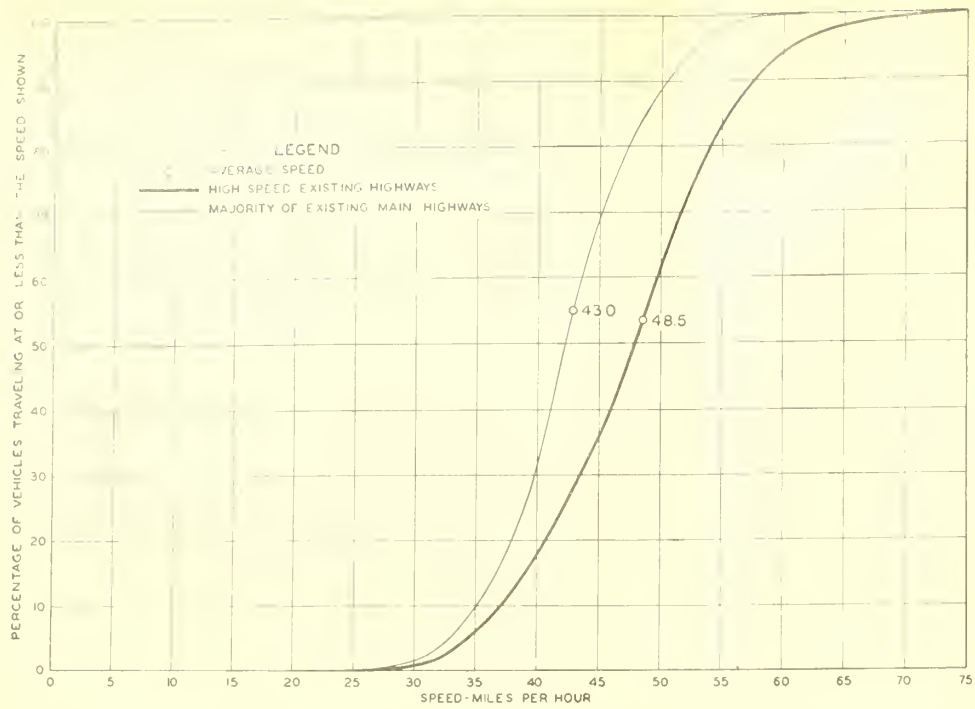


Figure 4.—Frequency distribution of travel speeds of free-moving vehicles on level, tangent sections of two-lane rural highways.

roadway condition that prevents vehicles from moving safely at a speed of 30 miles per hour lowers a roadway's capacity. A traffic density exceeding the critical density is one of these conditions.

Drivers' Desired Speeds

Drivers are influenced in the selection of their vehicular speeds by a large number of variables, such as the condition of the surface, the traffic density, the length of trip, the condition of their cars, and their own individual idiosyncrasies. **When traffic and roadway conditions are such that drivers can travel at whatever speed they desire, there is always a wide range in the speeds at which the various individuals operate their cars.** This fact is illustrated by the typical speed distribution curves shown in figure 4.

Average speeds higher than those shown in figure 4 have been recorded, particularly on rural multilane highways in the West and Midwest, and at one location an average speed of 58 miles per hour for passenger cars has been reported. **It is uncommon, however, for the average speed to exceed 50 miles per hour even during very light traffic densities.** Regardless of what the average speed may be, the general shape of the speed distribution curve for free-moving vehicles will be about the same as for the curves shown in figure 4. **It is evident from this information that comparatively few passenger-car drivers desire to travel at speeds which equal or approach the potential speeds of their vehicles even under the most favorable highway and traffic conditions.**

Effect of Traffic Volume on Speed

Prior to the development of equipment for accurately measuring the relation between speed and traffic volume, it was commonly assumed that at some traffic volume below the possible capacity of the street or highway a slight increase in traffic volume would cause a marked reduction in the average vehicle speed. Had this assumption been correct, it would have provided an ideal criterion by which to determine practical capacities. **Investigations conducted on an extensive scale have definitely shown, however, that there is a straight-line relation between traffic volume and average speed when other conditions are identical and the critical traffic density is not exceeded.** Figure 5 shows this relation for two typical rural conditions. The same relation between average speed and traffic volume is applicable to routes through cities, including streets with traffic signals, provided that the traffic densities do not exceed those that occur when the facility is accommodating its possible capacity and provided also that other influencing factors are constant.

When the average speed is reduced as a consequence of an increase in traffic volume, the speed distributions change in the manner shown by figure 6, which illustrates conditions observed on a typical high-speed highway. This set of curves shows that **when the distribution of speeds and the average speed are known for a particular traffic volume on a given route, it is possible to predict with a high degree of accuracy the speed distributions for other average speeds.**

Difference in Speed a Criterion of Possible Capacity

The range in the speeds of individual vehicles, at a point where the traffic flow is not interrupted by traffic signals, decreases with an increase in traffic volume as shown by figure 6. With little traffic on the highway, drivers travel at their desired speeds. As the traffic volume increases, individual drivers are affected more and more by other traffic. The high-speed drivers obviously are affected to a greater degree than the lower-speed drivers. **When the traffic volume becomes high enough to prevent the high-speed drivers from passing the low-speed drivers, all traffic must move at approximately the same speed and the average difference in speed between successive vehicles will approach or become zero. When this occurs, the traffic volume has reached the possible capacity of the particular facility under the prevailing conditions and the number of vehicles per unit length of highway will**

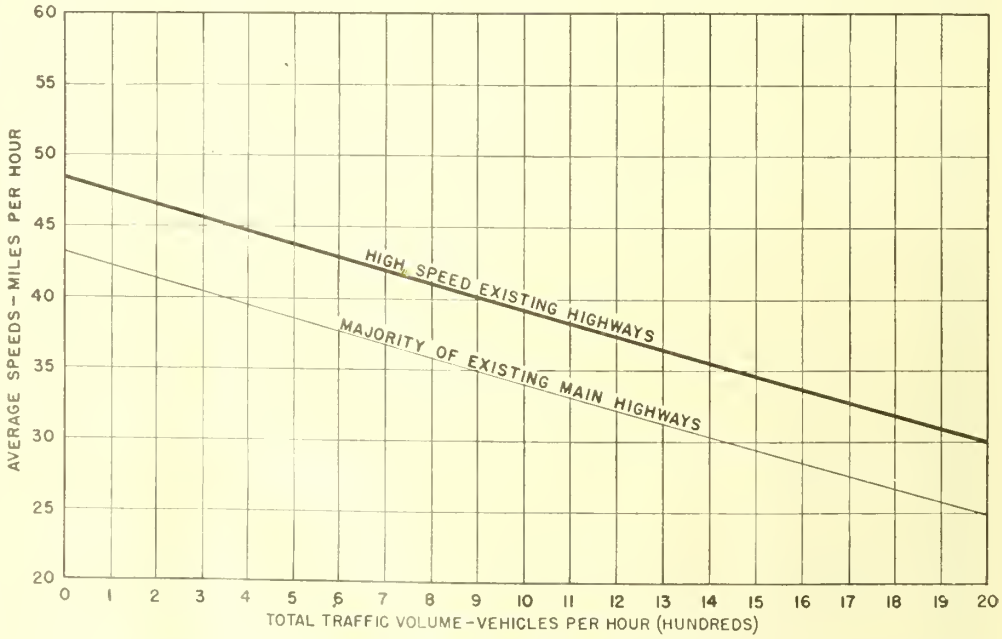

HIGH SPEED EXISTING HIGHWAYS
MAJORITY OF EXISTING MAIN HIGHWAYS
AVERAGE SPEEDS-MILES PER HOUR
TOTAL TRAFFIC VOLUME-VEHICLES PER HOUR (HUNDREDS)

Figure 5.—Average speeds of all vehicles on level, tangent sections of two-lane rural highways.

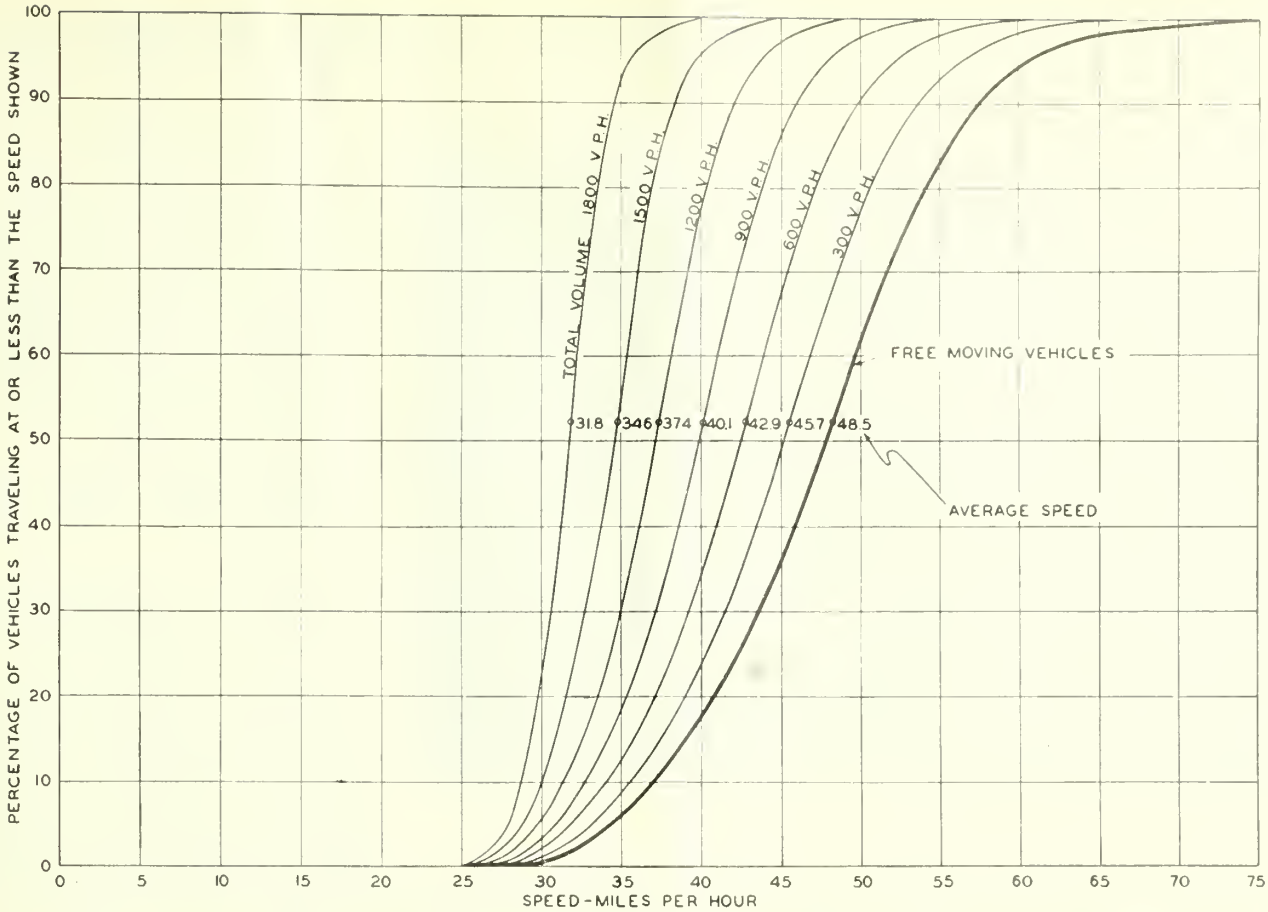


Figure 6.—Typical speed distributions at various traffic volumes on level, tangent sections of two-lane, high-speed existing highways.

then equal the critical density for that highway. If the traffic density continues to increase there will be a marked reduction in speed with the result that the traffic volume, which is the product of the density and the speed, will be lowered below the highway's possible capacity.

Extensive studies have shown that on facilities where drivers are not influenced by speed limits, **the average difference in speed between successive vehicles decreases as a straight line relation with an increase in traffic volume until the traffic volume reaches the facility's possible capacity at the critical density.** This finding has made it possible to determine the effect that various highway features and traffic conditions have on the possible capacities of different types of highway facilities. It is also possible to predict, with a high degree of accuracy, the capacity of a particular facility without waiting until that facility becomes congested. This is accomplished by plotting the average difference in speed during a very low traffic density and during a somewhat higher traffic density on a graph, using the traffic volume and the mean difference in speed between successive vehicles as the coordinate scales. A straight line going through these points and extended to the point at which the speed difference is zero will indicate the traffic volume representing the possible capacity.

Vehicles stopped or standing on the pavement not only deprive other traffic of a usable portion of the roadway, but by causing a reduction in speed of traffic they lower the capacity of the remaining lanes.

Turn-out lanes permit busses to stop with little or no interference with other traffic. This one might well have been a little wider.

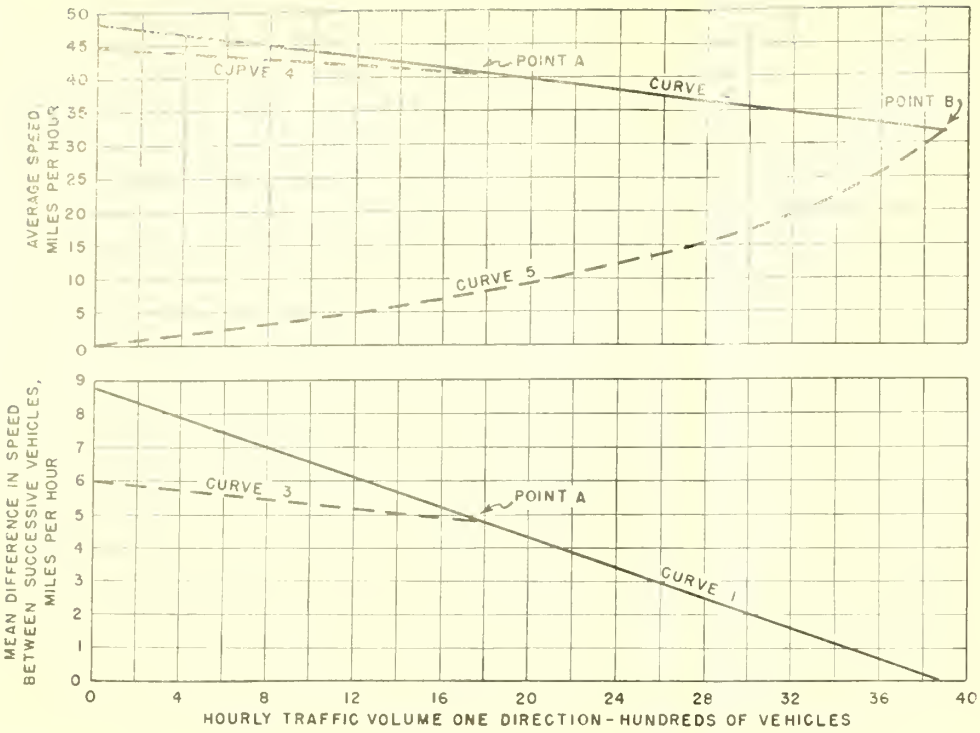


Figure 7.—Speeds and speed differences on a typical four-lane divided highway.

When calculating the mean difference in speed between successive vehicles, the speeds of individual vehicles are first separated by direction of travel and then arranged in the same order as that in which the vehicles passed a given point on the highway, regardless of the number of lanes for each direction of travel. The difference in speed is then calculated for pairs of vehicles using each vehicle first as the following vehicle and then as the leading vehicle. The differences are then averaged arithmetically, disregarding the algebraic signs.

For two-way, two- and three-lane roads, the means for the two directions are averaged by weighting according to the number of vehicles n each direction of travel. For multilane highways, the mean difference in speed for one direction of travel is never combined with the corresponding value for the other direction because the resulting figure would be meaningless.

Relation Between Speed and Traffic Volume

The cause of traffic "tie-ups" or the sudden occurrence of complete congestion on a facility can be explained by employing the average difference in speed in combination with the relation between speed and volume as shown by figures 3 and 5. The change in the average speed and the change in the mean difference in speed between successive vehicles with an increase in traffic volume on a typical four-lane divided highway without a speed limit are shown by the solid lines of figure 7. Curve 1 shows that the average difference in speed between successive vehicles decreases as a straight line with an increase in traffic volume until at a total volume in one direction of 3,900 vehicles per hour, the possible capacity of the two lanes, all vehicles are moving at the same speed.

Curve 2 shows that the average speed also decreases from 48.5 to 32 miles per hour with the traffic volume increasing to 3,900 vehicles per hour. On similar highways with an enforced speed limit of 50 miles per hour the average speeds and the mean difference between the speeds of successive vehicles for the low traffic volumes would be as shown by curves 3 and 4. That is, for volumes under 1,750 vehicles per hour, the speed limit would have a greater effect on speeds than would the traffic densities.

Possible Range of Speeds for Each Volume

Curve 5 is based on the combined volumes of the two curves shown in figure 3 for four-lane highways. It represents the lowest possible speed of traffic if the two lanes are to accommodate the various total traffic volumes. For example, with a total volume of 2,000 vehicles per hour in the one direction, the average speed of traffic could be anywhere between 10 and 40 miles per hour. With an average of 40 miles per hour, there would be a wide distribution of speeds, whereas if the average were 10 miles per hour all vehicles would be moving at approximately the same speed. Should some condition cause traffic to slow to less than 10 miles per hour, the traffic capacity of the highway would become less than 2,000 vehicles per hour and traffic would immediately become badly congested. The intersection of curve 5 with curve 2 at point *B* represents the maximum possible capacity of the roadway under the prevailing conditions and is the maximum volume that can occur with a zero difference in speed between successive vehicles. The distance between curves 2 and 5 is a direct measure of the range in average speeds that can occur at the different total traffic volumes. The various distances are therefore a relative measure of the safety factor present to prevent the occurrence of complete traffic stagnation. Any point representing the relation between speed and volume must fall within the area between these two curves.

Traffic Volume Below Possible Capacity When Critical Density is Exceeded

To understand why complete congestion so often occurs when a highway is operating at or near its possible capacity, assume that traffic on the road which figure 7 represents has gradually increased to a volume of 3,900 vehicles per hour at an average speed of 32 miles per hour. Assume also that this traffic volume is approaching a point on the highway where drivers suddenly reduce their speeds to 20 miles per hour because vehicles are entering from a side street. The capacity of the highway at the point where speeds are only 20 miles per hour cannot exceed 3,250 vehicles per hour as shown by curve 5. Under this condition, vehicles would immediately start to accumulate at this point on the highway at the rate of 650 per hour, causing a sudden increase in the density of traffic. If the approach volume of 3,900 vehicles per hour and the restricted condition existed for only a few seconds, some vehicles would be required to stop and the traffic volume would immediately drop to zero at this point on the highway, as shown by curve 5. The queue of vehicles at a standstill would continue to increase in length until the volume of approaching traffic dropped to no more than 3,250 vehicles per hour and probably not until the approaching volume had dropped to 3,000 vehicles per hour. (One vehicle every 2.4 seconds has been found to be the average rate at which consecutive vehicles stopped in line get started again when there is an open road ahead.)

Even though the cause of the restriction lasts but a few seconds or minutes, additional vehicles might continue to become stopped for a considerable time after the cause of the restriction had been removed. These vehicles would form a queue which would move down the highway in the direction opposite to that of traffic flow. Queues of vehicles at a standstill have been observed on the Oakland-Bay Bridge and the Pulaski Skyway several miles from the scene of the original restriction, even though traffic was apparently operating in its normal manner between the queue and the place where the queue originally started to form.

Part IV.—Roadway Capacities for Uninterrupted Flow

INTRODUCTION

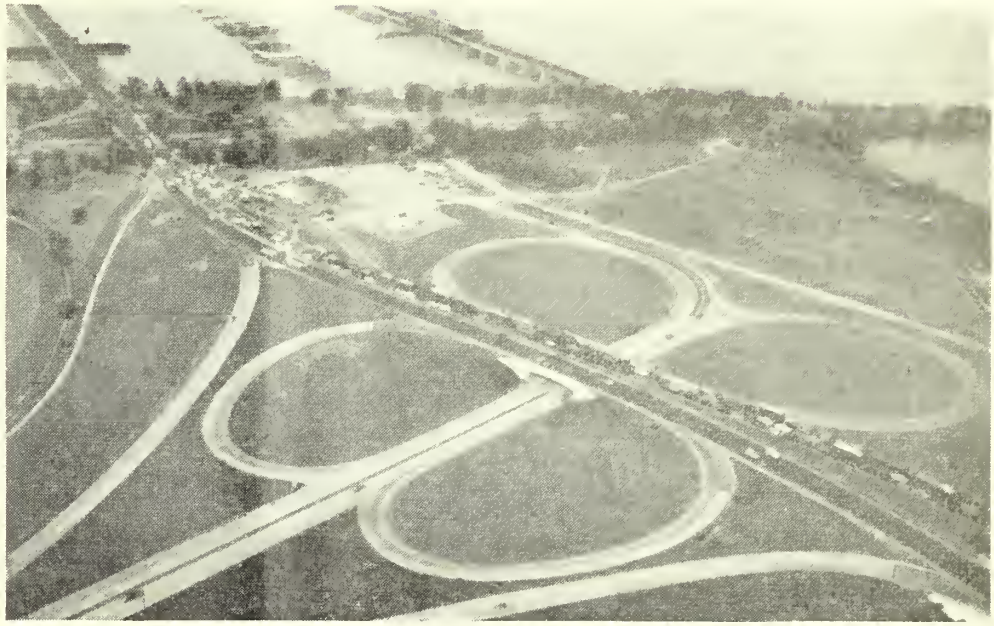
A logical sequence for presenting the available capacity material would consist of reporting the highest traffic capacities attainable under ideal conditions and then showing, by successive steps, the quantitative extent to which various traffic and roadway conditions affect these ultimate capacity values. That is the procedure followed in this chapter. Basic capacities are presented for the three general classes of highways: two-lane, three-lane, and multilane. Possible capacities are then discussed, and finally a suggested procedure is outlined for arriving at practical capacities for roads of all descriptions where the movement of traffic is uninterrupted by intersections or junctions.

As a source of information for this report, results are available from studies conducted by many individuals and organizations; in particular, those for the comprehensive studies of the dynamics of highway movement conducted by the Bureau of Public Roads in cooperation with State highway departments in all sections of the country. The data obtained and analyzed by the Bureau of Public Roads alone include detailed information such as individual vehicle speeds and spacings between vehicles under actual operating conditions at approximately 500 locations. During the periods of study at these locations traffic increased from comparatively low densities to the peak densities and then decreased to a low level again. By the use of special electro-mechanical equipment designed and built especially for this purpose, detailed data for slightly over a million vehicles have been recorded during the past 10 years for many of the most heavily traveled roads in the country.

The results of these studies have been supplemented by investigations of motortruck performance, including grade-climbing ability and braking or stopping distances; studies of the driving characteristics of various classes of motor-vehicle operators; studies of passing practices on two-lane roads in which detailed information was recorded for nearly 20,000 passing maneuvers; and studies of the effect on traffic behavior of various types of pavement markings and other control devices. These studies are in addition to the periodic speed studies made by the State highway departments, which have included speed observations on 2 million vehicles at 787 locations since 1939, and the traffic counts made manually and by hourly recording electrical counters for the purpose of obtaining annual and peak volumes on all main highways throughout the United States.

BASIC CAPACITIES

The uninterrupted flow of traffic may generally occur only on urban freeway facilities and on the sections of rural highway that are removed from the influence of intersections at grade. Even on these facilities, however, certain traffic and roadway conditions can cause an interruption of the normal traffic flow. It is nevertheless essential in a study of highway capacity to know the capacity of a facility for conditions of uninterrupted flow in order that proper deductions can be made for conditions that cause the flow to be interrupted.



The severe congestion here illustrated resulted from an emergency stop of one vehicle on the bridge in background.

Multilane Roads

The largest number of vehicles that can pass a point one behind the other in a single traffic lane, under the most ideal conditions that can possibly be attained, is between 2,000 and 2,200 passenger cars. So far as is known, hourly lane volumes within this range have been recorded at not more than two locations. Volumes only slightly over 2,000 vehicles per hour per lane have been recorded at several locations, but all reports of lane capacities in excess of 2,300 vehicles per hour have proved to be incorrect.

Traffic volumes in the neighborhood of 2,000 vehicles per lane per hour can occur only if the following five conditions are satisfied:

1. There are at least two lanes for the exclusive use of traffic traveling in one direction.
2. All vehicles move at approximately the same speed, each driver being restricted to the speed of the vehicle ahead. This speed, which is governed by the speed of the slowest drivers, must be between 30 and 40 miles per hour.
3. There are practically no commercial vehicles.
4. The width of traffic lanes, shoulders, and clearances to vertical obstructions beyond the edge of traffic lanes are adequate.
5. There are no restrictive sight distances, grades, improperly superelevated curves, intersections, or interferences by pedestrians.

Since these conditions can be satisfied on a multilane facility, the maximum number of passenger cars that can pass a given point during 1 hour under the most nearly ideal roadway and traffic conditions which can possibly be attained, or **the basic capacity of multilane roads, is 2,000 passenger cars per lane per hour.**

It has been rather widely assumed that the capacity of multilane facilities decreases with an increase in the number of lanes. This is not necessarily true. The Outer Drive in Chicago, for example, with its eight lanes, is one of the most efficient facilities. It has accommodated an average of 1,958 vehicles per lane per hour in the one direction of travel during several periods when manual counts were being conducted. The provision of adequate access and egress facilities such that the full capacity of all traffic lanes can be utilized, however, becomes increasingly difficult as the total number of lanes is increased.

Three-lane roads

On two- and three-lane roads there are not two lanes for the exclusive use of traffic traveling in the one direction. The character of operation is therefore entirely different on these roads from that on multilane highways, since vehicles performing passing maneuvers are forced to use a traffic lane that is provided for vehicles traveling in the opposite direction. Consequently, the basic capacities of two-

and three-lane [illegible] to vet than for multilane facilities.

When traffic on a two-lane highway is evenly divided by directions, when there are no restrictive sight distances, and when other conditions are ideal, vehicles can completely fill the two outside lanes by utilizing the center lane for passing, thus filling the long gaps that would otherwise occur between vehicles in the outside lanes ahead of slow-moving vehicles. Likewise, it is only when there is little or no traffic in the one direction that the one outside lane and the center lane can become filled with vehicles traveling in the other direction so that the characteristics of the flow may become similar to those in one direction on a four-lane highway. **The basic capacity of a three-lane, two-way road is therefore 4,000 passenger cars per hour or an average of 1,333 passenger cars per lane per hour. The basic capacity for one direction is limited to 2,000 passenger cars per hour on any section with but a single restrictive sight distance.**

Extensive studies have shown that there is no basis for the common assumption that most efficient operation on a three-lane road will be realized with approximately two-thirds of the traffic in one direction. Facts show that the high volumes can be handled most efficiently when traffic is evenly divided by direction, especially when there are passing sight-distance restrictions. As yet, the Committee has been unable to locate any count approaching 4,000 vehicles per hour on a three-lane road, and this is probably because there are sufficient sight-distance restrictions on most existing three-lane roads to prevent their basic capacities from being attained. A total count of 3,064 vehicles per hour has, however, been recorded on a three-lane road in New Jersey.

Two-Lane Roads

On a two-lane, two-way road, vehicles must, to overtake and pass vehicles traveling in the same direction, use the lane normally used by oncoming traffic. With few or no vehicles traveling in the one direction and no restrictive sight distances, traffic in the other direction can keep one lane completely filled because gaps that occur ahead of the slower-moving vehicles can almost immediately be filled by vehicles performing passing maneuvers. Whenever one vehicle traveling in the direction of the light flow appears, however, all vehicles traveling in the direction of the heavy movement must crowd into one traffic lane. Since the capacity of one traffic lane is limited to a maximum of about 2,000 passenger cars per hour, the basic capacity of the two-lane road under these conditions would be 2,000 passenger cars per hour.

With traffic evenly divided by directions, opportunities to overtake and pass slow-moving vehicles are sufficiently restricted by oncoming traffic to limit the flow in each direction to 1,000 passenger cars per hour. At this traffic volume, spaces occur ahead of the slow-moving vehicles which cannot be filled by other vehicles performing passing maneuvers. In effect, traffic in both directions tends to form queues which continue to increase in length until the spaces between the queues become sufficiently long to permit the performance of passing maneuvers. As soon as a few passing maneuvers are performed, the spaces between the queues become partially occupied and are no longer of sufficient length for the performance of passing maneuvers, and the queues immediately start forming again.

This accordion effect, with all but a very limited number of vehicles traveling at the same speed as the vehicle immediately ahead, occurs at the same total traffic volume regardless of the distribution by directions. **The basic capacity of a two-lane, two-way road is therefore a total of 2,000 passenger cars per hour regardless of the distribution by directions.**

Total traffic volumes close to 2,000 vehicles per hour have been recorded on two-lane, two-way facilities during periods when most of the traffic traveled in the one direction and also during periods when the traffic was evenly divided by direction. In only three cases have two-lane, two-way facilities been reported to have accommodated somewhat more than 2,000 vehicles in 1 hour. These occurred at tunnels where the comparatively short two-lane roadway sections were bottlenecks between higher-capacity facilities.

POSSIBLE CAPACITIES

When roadway and traffic conditions are ideal, the possible capacity of a facility with uninterrupted flow corresponds to its basic capacity. **Since roadway and traffic conditions are seldom ideal, the possible capacity of a facility, or the maximum number of vehicles that can pass a given point on a lane or roadway during 1 hour under the prevailing roadway and traffic conditions, is generally lower than its basic capacity.**

In practice, the only application of a roadway's **basic capacity** is that it forms a starting point from which its possible capacity can be estimated or calculated by deducting the effect of the prevailing conditions which are not ideal, whereas the **possible capacity** is a positive quantity that has a direct application to many problems with which the engineer is faced.

In the discussion of practical capacities, there are listed a number of the more important factors that influence traffic capacities when the flow is uninterrupted. **The resulting effect that these factors have on traffic capacity must be subtracted from the basic capacities to obtain the possible capacities for the prevailing conditions.**

PRACTICAL CAPACITIES

All vehicles must move at approximately the same speed when a street or highway is operating at its possible capacity, each driver being restricted by the speed of the vehicle ahead, which is usually the speed of the slowest group of drivers. Under this condition most drivers consider a highway extremely congested, because most of them want to travel faster than the slowest group, and they desire some freedom in the selection of their individual speeds. The average vehicle speed on a highway operating at or near its possible capacity is lower than the speed which most drivers consider reasonable.

Criteria of Practical Capacities Under Ideal Conditions

The practical capacity of a highway carrying an uninterrupted flow of traffic is reached when a higher volume will cause drivers to be unreasonably restricted. "Unreasonably re-

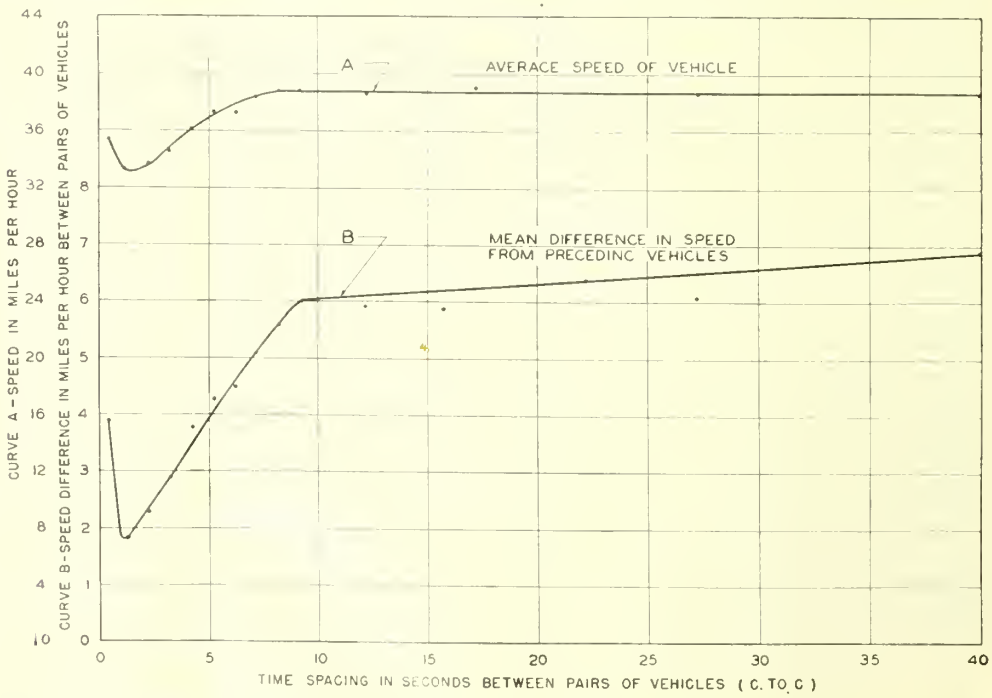


Figure 8.—Speed characteristics of vehicles traveling at given time spacings behind preceding vehicles.

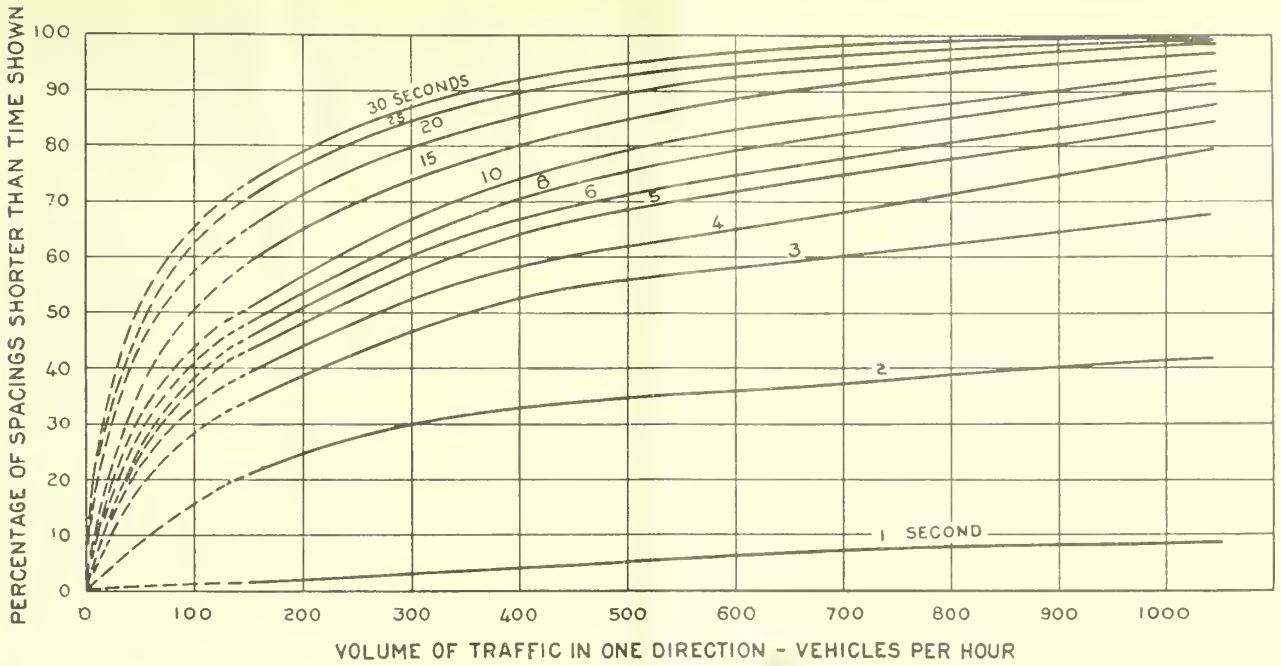


Figure 9.—Frequency distribution of time spacings between successive vehicles, at varying volumes of traffic on a typical two-lane rural highway.

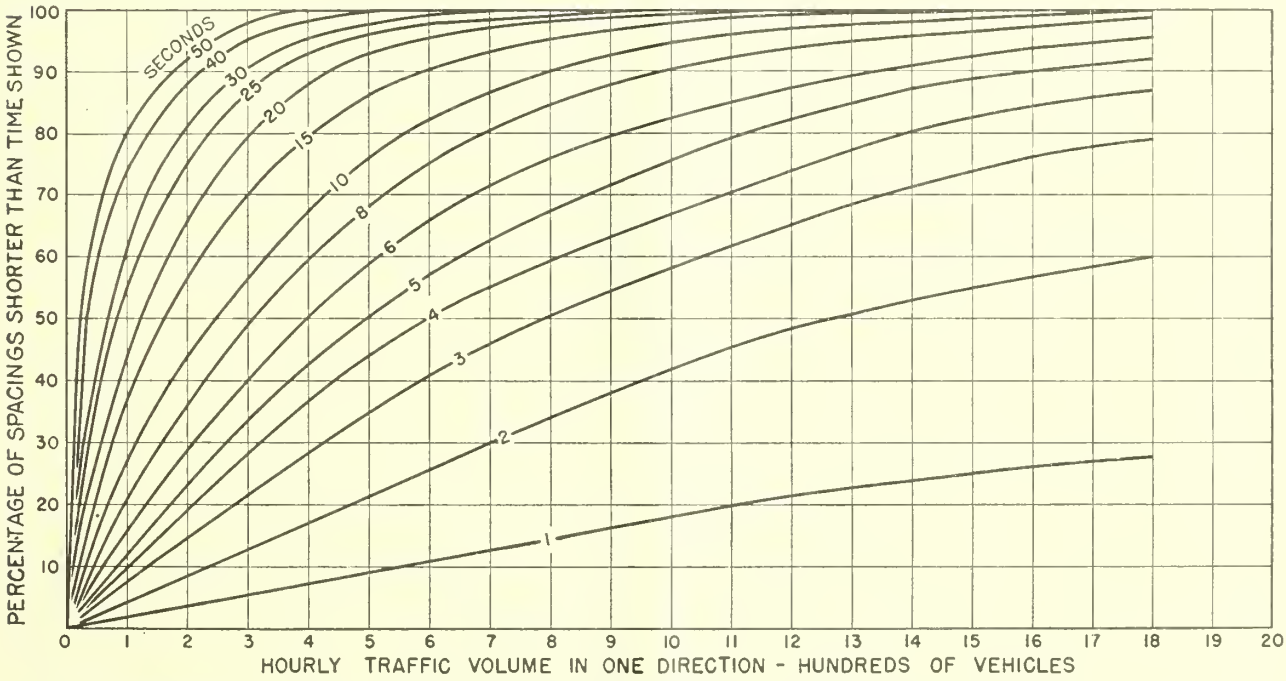


Figure 10.—Frequency distribution of time spacings between successive vehicles traveling in the same direction, at various traffic volumes on a typical four-lane rural highway.

stricted" is only a relative term which will vary for different locations. In urban areas, for example, a driver is willing to accept the regulation of his speed by other traffic to a greater extent than in rural areas, so long as he can keep moving at a speed he considers reasonable under the circumstances. Furthermore, **the relative restriction and the average travel time will increase uniformly as the traffic volume on a highway is increased from low volumes to the maximum possible capacity of the highway. There is no volume, below the highway's maximum capacity, at which a further increase in volume will cause a marked increase in congestion or decrease in speed.** This has been illustrated by figures 5 and 7. It is necessary, therefore, to apply other criteria which have been developed to measure congestion in the selection of practical capacities for various conditions.

Vehicle spacings

The most sensitive index of traffic congestion on two-lane highways is shown in figure 8 (curve *B*). This figure shows that as the spacing between vehicles decreases there is little change in the difference in speed of successive vehicles until the time spacing is reduced to 9 seconds. Below that spacing, the speed of the following vehicle approaches that of the one ahead very rapidly, indicating that with a spacing of 9 seconds, drivers are affected by the presence of the car ahead, and that the shorter these spaces become the greater is the effect.

If all vehicles using a highway were equally spaced, determination of the point of incipient congestion would be a simple matter. However, it is a well-known fact that vehicles do not move with uniform separating intervals, but rather that they tend to form groups. Studies made on tangent highway sections in

A section of two-lane road where the design features are ideal for high capacity.

many parts of the country show that this tendency to form groups is remarkably uniform regardless of the traffic volume. Figures 9 and 10 are typical of the vehicle-spacing distributions for conditions where the flow of traffic is uninterrupted.

Under nearly any conditions of speed and traffic volume, approximately two-thirds of the vehicles will be spaced at, or less than, the average distance between vehicles. Using figure 9, the curve of vehicle spacings on two-lane roads, it will be found, for example, that with an hourly volume of 180 vehicles in one direction (representing an average spacing of 20 seconds) about 120 vehicles will be 20 seconds or less behind the car ahead, and of these about 90, or 52 percent of the total, will be spaced at 9 seconds or less and will therefore be affected to some degree by the car ahead. As the traffic volume increases, the number so spaced increases. With 200 vehicles per hour in one direction, 55 percent will feel some effects of congestion; with 300 vehicles per hour the figure becomes 65 percent; with 400 it becomes 72 percent, and so on until over 90 percent of the vehicles are affected when the volume becomes 1,000 vehicles per hour in each direction.

If it be considered that a highway is congested when 72 percent of the drivers must govern their speeds by the speeds of other vehicles, 800 vehicles per hour is the practical capacity of a two-lane road.

Passing opportunities

Another index of congestion is the availability of opportunities for vehicles to overtake and pass slower vehicles moving in the same direction. If the alinement of a highway is not a factor, passing on a two-lane road is limited only by the time the lane normally used by oncoming traffic is occupied. The ratio of the number of passings required per mile of highway for drivers to maintain their desired speeds, to the number of passings that they can actually perform, is a measure of traffic congestion. Figure 11 compares the desired number of passings with the actual number that can be performed under typical conditions on a two-lane road with tangent alinement.

The total number of passings required for all drivers to maintain their desired speed increases as the square of the traffic volume. Actually, however, the total number of passings that occur increases with an increase in the total traffic volume up to 1,300 vehicles per hour and then decreases rapidly. To maintain his free speed, the number of passings each driver would make increases directly as the traffic volume increases. Actually, however, **the number of passings made by the average driver increases as the density increases up to 800 vehicles per hour, remains about the same between 800 and 1,200 vehicles per hour, and thereafter decreases with a further increase in the traffic density. The fact that the average driver on a two-lane, tangent highway should increase the number of passings he makes as the traffic volume goes above 800 vehicles per hour, but can make no material increase due**

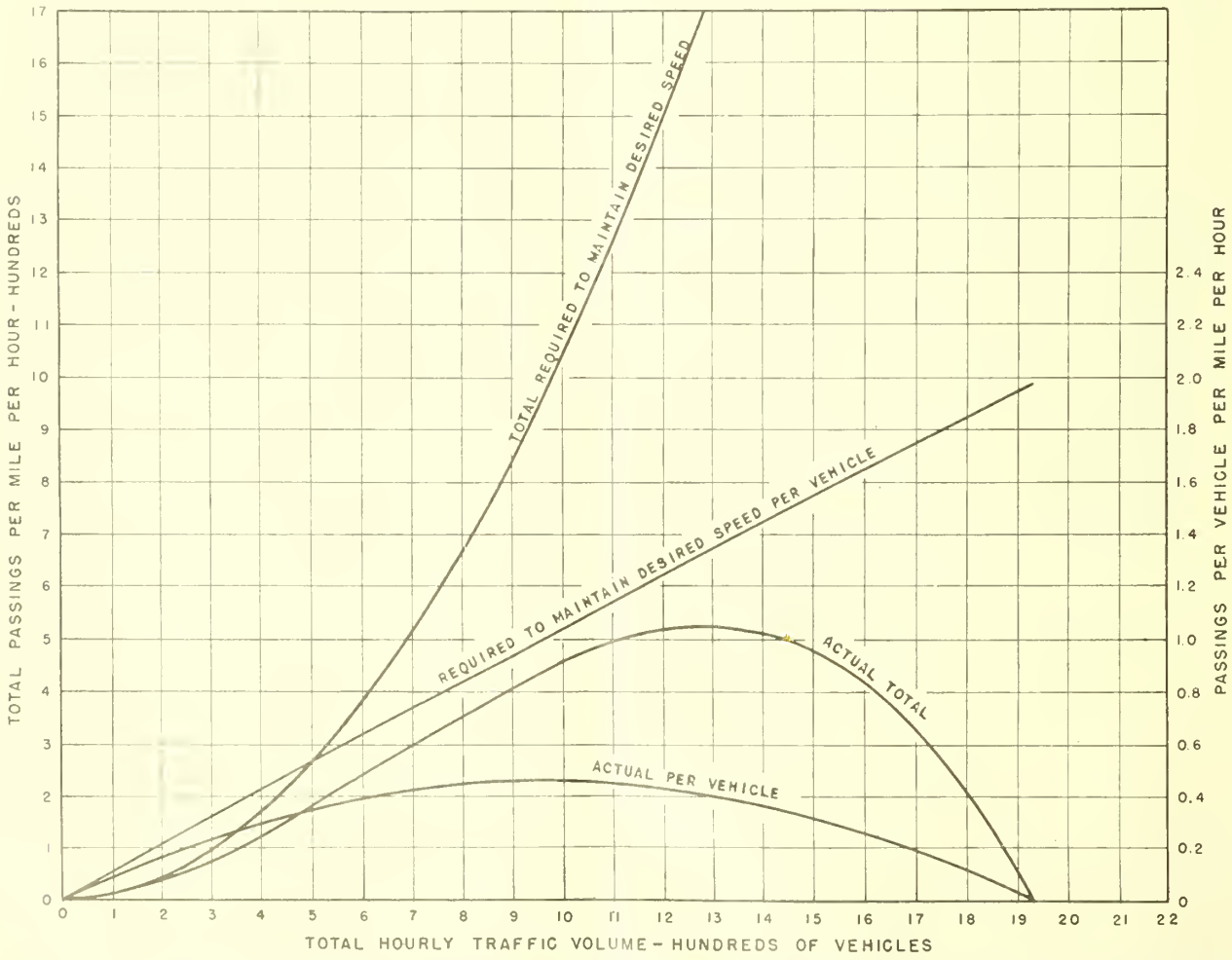

TOTAL PASSINGS PER MILE PER HOUR - HUNDREDS
PASSINGS PER VEHICLE PER MILE PER HOUR
TOTAL HOURLY TRAFFIC VOLUME - HUNDREDS OF VEHICLES
TOTAL REQUIRED TO MAINTAIN DESIRED SPEED
REQUIRED TO MAINTAIN DESIRED SPEED PER VEHICLE
ACTUAL TOTAL
ACTUAL PER VEHICLE

Figure 11.—Comparison of actual number of passings performed and number that would be required at various traffic volumes for all vehicles to maintain their free speed on a two-lane highway with two-thirds of the total traffic in one direction.

to the traffic density, is a very important criterion in the determination of practical capacities for two-lane highways. (A traffic volume of 800 vehicles per hour with the normal percentage of commercial vehicles is equivalent to 900 passenger cars per hour.)

Operating speeds

The most significant index of traffic congestion during different traffic volumes, as far as drivers are concerned, is the over-all speed (exclusive of stops) which an average motorist can maintain when trying to travel at the highest safe speed. This over-all speed is termed the "operating speed."

A driver on a two-lane, tangent highway can travel at a uniform speed of say 50 miles per hour, until he overtakes a slower-moving vehicle. He would then either pass the slower vehicle, if there was no oncoming traffic, or he would slow down to the speed of the vehicle ahead and wait until the opportunity to pass presented itself. For a driver to travel at a uniform speed of 50 m.p.h. on a modern two-lane highway carrying a total of 300 vehicles per hour equally divided in each direction, he would be required to pass an average of 24 vehicles per hour. At a volume of 900 vehicles per hour he would be required to pass 130 vehicles per hour. In the first case, 43 percent of the vehicles he passed would be traveling at speeds exceeding 40 m.p.h. while in the second case only 23 percent of the vehicles he passed would be exceeding 40 m.p.h.

Under actual operating conditions on a two-lane highway, the left lane would not always be free of oncoming traffic at the time the driver overtook a vehicle traveling at a slower speed, so even on highways with no sight-distance restrictions, he would sometimes be forced to reduce his speed and await an opportunity to pass. This delay would cause a decrease in his average speed which in turn would decrease the number of vehicles overtaken within a given time period.

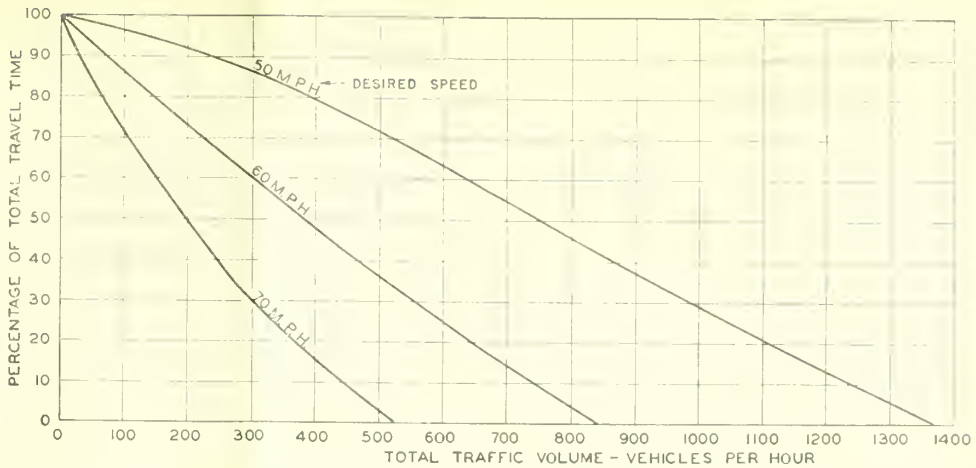


Figure 12.—Percentage of time that desired speed can be maintained on two-lane, level highway with no restrictive sight distances.

Figure 12 shows the portions of the total time that a driver desiring to travel at a certain speed can travel at that speed on a two-lane highway with no sight-distance restrictions. A driver desiring to travel at 70 m.p.h. can travel at this speed 100 percent of the time as long as there is no other traffic on the highway. However, when there is other traffic on the highway, he soon overtakes another vehicle traveling at a slower speed and, unless there is a space between vehicles in the opposing traffic lane of sufficient length to permit him to pass, he must reduce his speed until such an opportunity does occur, after which he can again increase his speed to 70 m.p.h. At a total traffic volume of 200 vehicles per hour, it will be possible for the driver to travel at 70 m.p.h. 50 percent of the time if he takes full advantage of his opportunities to pass the slower drivers. At a traffic volume of about 550 vehicles per hour he will not be able to travel at 70 m.p.h. any of the time because after passing one vehicle he must start slowing down to avoid a rear-end

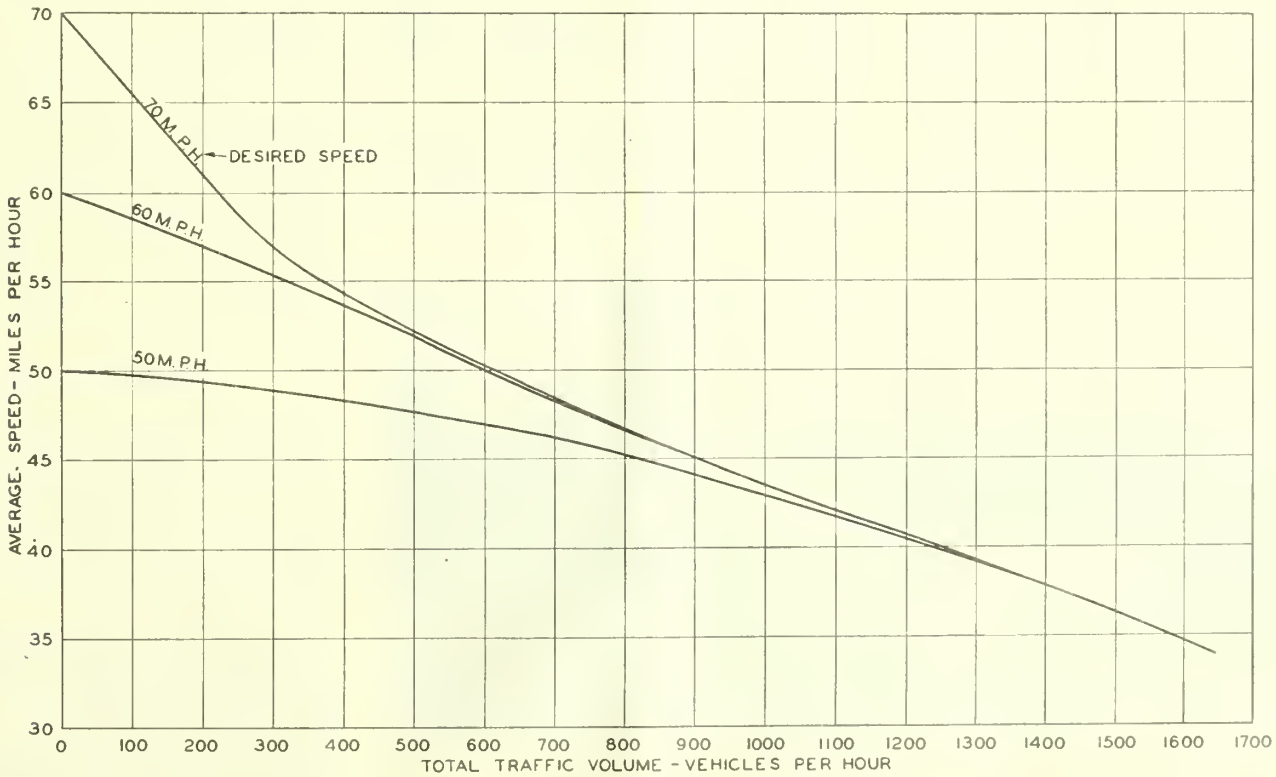


Figure 13.—Average speeds for drivers traveling at their desired speeds whenever possible on a two-lane, level highway with no restrictive sight distances.

collision with the preceding vehicle traveling in the same direction or a head-on collision with oncoming traffic in the opposing traffic lane.

Likewise, a driver trying to travel 60 m.p.h. can maintain this speed 100 percent of the time when there is no other traffic, and 50 percent of the time when the traffic volume is 400 vehicles per hour, but he cannot at any time attain this speed when the traffic volume is in excess of 850 vehicles per hour. Also, a driver trying to maintain a speed of 50 m.p.h. can accomplish this 50 percent of the time with a traffic volume of 750 vehicles per hour but at no time can he travel at this speed when the traffic volume exceeds 1,400 vehicles per hour.

These curves in figure 12 also show the percentage of time that drivers must travel at various speeds below their desired speeds. For example, with a traffic volume of 400 vehicles per hour, a driver whose desired speed is 70 m.p.h. can travel at this speed 15 percent of the time; between 70 and 60 m.p.h., 35 percent of the time; between 60 and 50 m.p.h., 30 percent of the time; and is required to reduce his speed below 50 m.p.h. about 20 percent of the time.

A driver trying to go 70 m.p.h. can travel no faster than a driver trying to go 60 m.p.h. when the traffic volume exceeds 850 vehicles per hour. The 60- and 70-m.p.h. drivers can go no faster than the 50-m.p.h. driver when the traffic volume exceeds 1,400 vehicles per hour.

Effect of other traffic

Figure 13 shows the effect of other traffic on the over-all speed of drivers trying to maintain a certain desired speed. On tangent sections of existing high-speed, two-lane highways, a driver trying to maintain a speed of 70 m.p.h. without exceeding this speed finds that his over-all speed decreases rapidly with an increase in the traffic density. At a traffic volume of 200 vehicles per hour his over-all speed is reduced from 70 to 61 m.p.h. due to delays while waiting to pass slower-moving vehicles, and at 850 vehicles per hour his over-all speed will be 46 m.p.h. or the same as that for a driver trying to maintain a speed of 60 m.p.h. For all practical purposes the speeds for the 60- and 70-m.p.h. drivers are the same when the traffic volume is in excess of 300 vehicles per hour.

From these curves (fig. 13) it is possible to determine the hourly volume of traffic that can be accommodated by a two-lane highway at any specified operating speed when sight-distance restrictions are not a factor. If, for example, a two-lane road must accommodate peak volumes of 1,000 vehicles, the operating speed on that two-lane road, if it has no sight-distance restrictions, will be about 43 m.p.h. during periods that the peak volumes occur. It is possible from the results of these traffic studies to predict with a high degree of accuracy the operating conditions that will prevail with any given traffic volume on a highway with two, three, or four lanes where the flow is uninterrupted and other conditions are ideal. To determine the practical capacity of a facility it is necessary, first, to determine the operating conditions that the majority of motorists will accept as satisfactory. Then it is necessary to ascertain the highest standard of highway improvement that the governmental jurisdiction can support. Finally, it becomes necessary to reconcile the demands of the motorist with the means available for meeting these demands. Thus, in the final analysis, the matter of specifying precise values for practical highway capacities becomes a localized problem. **The Committee considers that it is of prime importance to relate traffic volumes accurately to operating conditions that will prevail so that individual agencies, with a thorough knowledge of the specific conditions, can decide on the most practical volumes to expect a facility to handle. The Committee also believes, however, that it is desirable to suggest practical capacities based on the normal desires of drivers under certain conditions.**

On most main rural highways, operating conditions are considered satisfactory for the average driver when the operating speed is 45 to 50 miles per hour during all but a few of the peak volume periods in a year.

With this operating speed, the average speed of all vehicles will be 40 to 45 miles per hour and at any one instant about 70 percent of the drivers will feel some effect of congestion, but they will have an opportunity to pass the slower-moving vehicles without unreasonable inconvenience. In certain sections of the country where drivers rarely experience congested conditions or where most trip lengths are relatively long, and on toll roads or other special high-type rural facilities, drivers might consider a highway unreasonably congested when those who so desire could not average 50 to 55 miles per hour during all but a few of the periods of peak volume in a year.

On urban facilities with uninterrupted flow, an operating speed of 35 to 40 miles per hour, resulting in an average speed for all traffic of 30 to 35 miles per hour, is considered reasonable.

Two-lane Roads

Under ideal roadway and traffic conditions a two-lane road where sight distanees are not restrictive will accommodate 900 passenger cars per hour and still permit operating speeds of 45 to 50 miles per hour. The corresponding figure for an operating speed of 50 to 55 miles per hour is 600 passenger cars per hour. Studies have shown that the distribution of traffic by directions on a two-lane road has practieally no effect on operating speeds or on the average interference between vehicles and relative congestion which the average driver experiences. **The maximum practical capacity of a rural two-lane road with uninterrupted flow is therefore a total of 900 passenger cars per hour regardless of the distribution of traffic by directions.**

Within urban areas or for access connections between industrial plants and the main highways, where operating speeds of 35 miles per hour are satisfactory, the practical capacity of a two-lane road would be 1,500 passenger cars per hour. At certain locations where operating speeds of 50 to 55 miles per hour are required, the practical capacity of a two-lane road would be 600 passenger cars per hour.

Multilane Roads

A multilane highway will provide the same or comparable operating conditions with 1,000 passenger cars per hour per lane in the direction of the heavier traffic flow as a two-lane highway with no restrictive sight distances operating at a total traffic volume of 900 passenger cars per hour. **The maximum practical capacity of a multilane rural highway with uninterrupted flow is therefore 1,000 passenger cars per lane per hour in the direction of the heavier flow.**

A multilane highway with uninterrupted flow will provide an operating speed of 35 to 40 miles per hour when the traffic volume in the one direction of travel is 1,500 passenger

A badly congested multilane highway. Cars in the second lane are here delayed by a vehicle desiring to make a left turn. Uncontrolled right of access has resulted in the conversion of this section of highway to the equivalent of one continuous intersection.

cars per lane per hour. **The maximum practical capacity of multilane freeways in urban areas, when access and egress facilities are not a factor, is 1,500 passenger cars per lane per hour in the direction of the heavier flow.** At this volume, drivers who so desire can safely maintain an over-all speed of 35 to 40 miles per hour, although the average speed of all vehicles will be 30 to 35 miles per hour. Also, exceptionally high volumes that occur frequently for short periods can be handled without complete congestion. There is also further significance in 1,500 vehicles per lane per hour as the maximum practical capacity of multilane roads in that this is the highest rate at which vehicles, after being once stopped, can pass a point in a single line.

Three-Lane Roads

The place of the three-lane, two-way pavement in the highway system has long been the subject of discussion. Three-lane highways have been constructed to accommodate traffic volumes in excess of those that can be handled efficiently by two-lane highways but that are not sufficiently great to require a four-lane road. They have also been built as stage construction projects in the development of four-lane undivided highways. The belief has been expressed, however, that traffic volumes justifying more than two lanes have by their natural growth soon become sufficient to congest a three-lane width. Furthermore, by the time a third lane can be planned and constructed on an already congested two-lane road, the traffic demand requires most of the increased capacity which the third lane provides, and thus the added lane provides only temporary relief.

With the rather recent universal change in favor of four-lane divided rather than undivided highways, there has been a marked tendency toward the elimination of three-lane road construction by going directly to a four-lane divided highway for traffic volumes in excess of those that can be accommodated by a two-lane road. A three-lane road does not lend itself to the ultimate development of a four-lane divided highway.

Hazard to traffic

There have been wide differences of opinion regarding the hazard to traffic on three-lane highways. From the results of the most comprehensive studies of accident rates of three-lane highways as compared with other types, there has, however, been a general acceptance of the belief that a traffic volume greater than can be accommodated by a two-lane highway justifies the added safety that can be provided by a four-lane divided highway.

From the point of view of driver behavior, the three-lane highway suffers a psychological disadvantage which might well result in an abnormally high accident rate. On a two-lane road, a driver engaged in a passing maneuver must encroach upon the left lane which is definitely provided for traffic in the opposite direction. He does this with full realization that his passing is accomplished only in the face of the superior rights of drivers in the opposing lane. In the case of the three-lane road, particularly with traffic evenly divided in the two directions, there is no clear-cut right-of-way distinction. A vehicle moving in one direction has as much right in the center lane as one moving in the other direction, and passings may involve much greater traffic hazards.

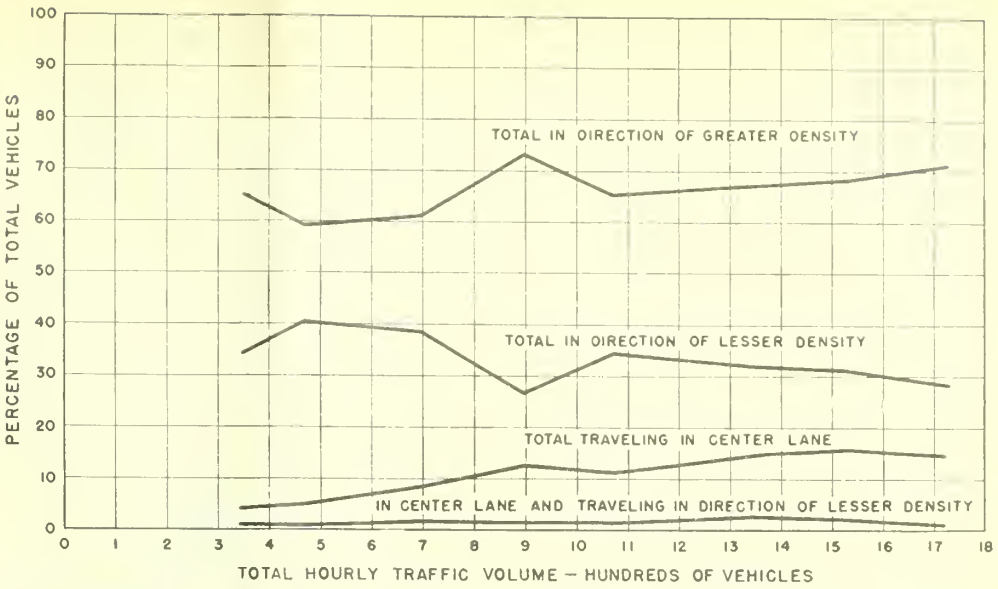

PERCENTAGE OF TOTAL VEHICLES
TOTAL IN DIRECTION OF GREATER DENSITY
TOTAL IN DIRECTION OF LESSER DENSITY
TOTAL TRAVELING IN CENTER LANE
IN CENTER LANE AND TRAVELING IN DIRECTION OF LESSER DENSITY
TOTAL HOURLY TRAFFIC VOLUME — HUNDREDS OF VEHICLES

Figure 14.—Distribution of vehicles between lanes on a level, tangent, three-lane highway.

Efficiency

It is commonly assumed that a three-lane highway is more efficient for locations where at least two-thirds of the traffic travels in one direction during high volume periods. This assumption is based on the idea that traffic traveling in the direction of the heavier volume will use two lanes, and traffic in the other direction will use one lane. Based on data obtained on modern three-lane highways with good alinements and profiles, figure 14 shows the percentage of vehicles that use each lane when approximately two-thirds of the traffic is in one direction. The percentage of vehicles traveling in the center lane at any one place increases as the total volume increases to 1,500 vehicles per hour. At this volume, only 15.9 percent of the vehicles were in the center lane, 13.8 percent traveling in one direction and 2.1 percent in the other. One driver had the same right as another to use the center lane of a three-lane highway and, regardless of the opposing traffic volume, he apparently exer-

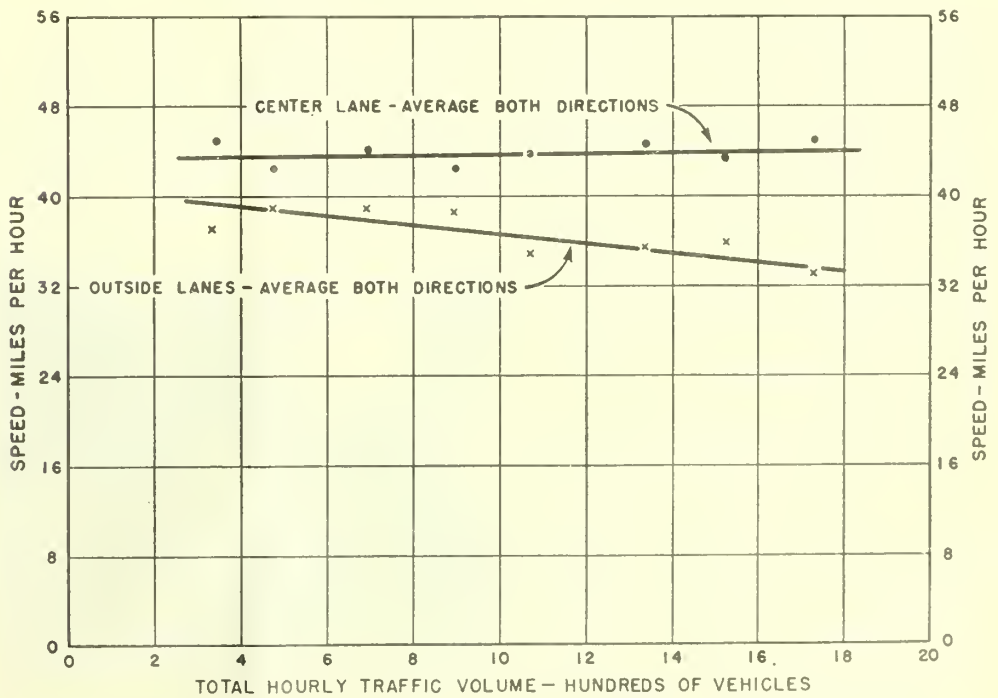

CENTER LANE - AVERAGE BOTH DIRECTIONS
OUTSIDE LANES - AVERAGE BOTH DIRECTIONS
SPEED - MILES PER HOUR
TOTAL HOURLY TRAFFIC VOLUME — HUNDREDS OF VEHICLES

Figure 15.—Vehicle speeds in center and outside lanes on a three-lane highway.

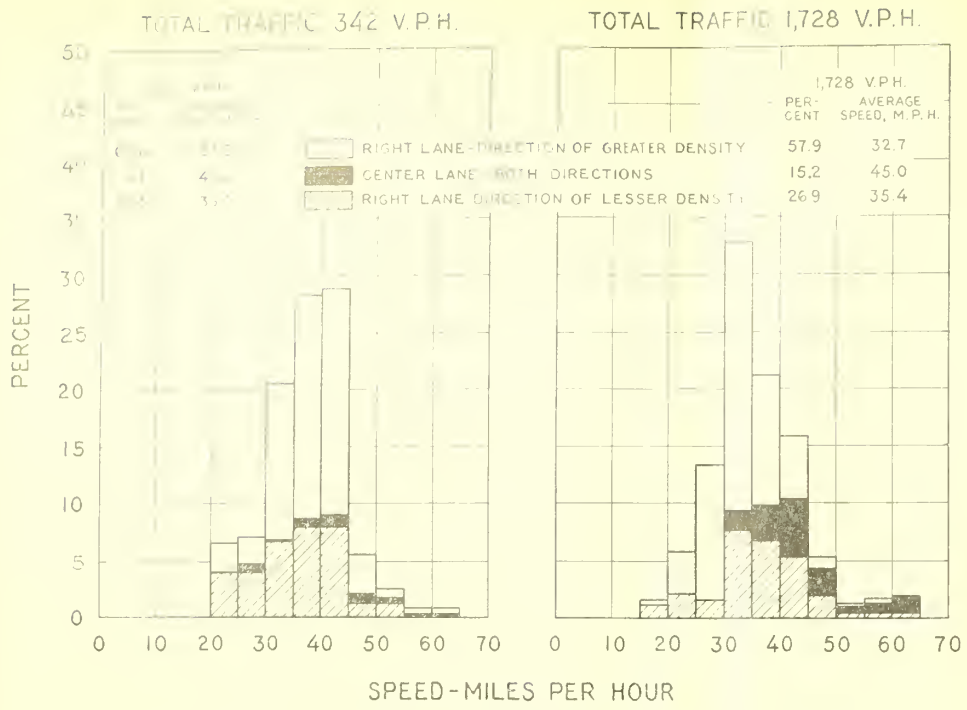

TOTAL TRAFFIC 342 V.P.H.
TOTAL TRAFFIC 1,728 V.P.H.
1,728 V.P.H.
PER-CENT
AVERAGE SPEED, M.P.H.
RIGHT LANE-DIRECTION OF GREATER DENSITY 57.9 32.7
CENTER LANE BOTH DIRECTIONS 15.2 45.0
RIGHT LANE DIRECTION OF LESSER DENSITY 26.9 35.4
PERCENT
0 5 10 15 20 25 30
0 10 20 30 40 50 60 70
SPEED-MILES PER HOUR

Figure 16.—Frequency distribution of vehicle speeds on a three-lane highway.

cised that right. An estimated maximum of about 300 vehicles per hour, or 15 percent, used the center lane at a volume of 2,000 vehicles per hour, which is well above the practical capacity of a three-lane road.

Figure 15 shows average speeds on three-lane highways for vehicles in the center and outside lanes, at different total volumes. There is a marked decrease in the average speed of vehicles in the right-hand lanes with an increase in the total volume, whereas the average speed for vehicles in the center lane does not change as the volume increases. Figure 16 further illustrates the tendency of drivers to travel at high speeds when using the center lane of a three-lane highway. This figure shows the distribution of speeds by lanes and direction at a low and a high total traffic volume. Although only 15.2 percent of the vehicles were in the center lane at the higher traffic volume, 41.7 percent of those traveling over 40 miles per hour, 58.3 percent of those traveling over 45 miles per hour, and 72.7 percent of those traveling over 50 miles per hour were in the center lane. This tendency of drivers to travel in the center lane of a three-lane highway during heavy traffic volumes at speeds nearly as high as those on four-lane divided highways during low traffic volumes, creates a hazardous condition and contributes to the severity of accidents. On two-lane highways and four-lane divided and undivided highways, drivers tend to reduce their speeds with an increase in the volume of traffic, regardless of the lane they occupy.

A similar analysis for periods when traffic was almost evenly divided by direction showed that under this condition a three-lane highway will handle traffic more efficiently than when the flow is predominantly in the one direction. Apparently the assumption of most efficient operation with two-thirds of the traffic in one direction is a figment of the imagination that cannot be substantiated by facts. Were it true, this assumption could be widely applied because it is the rule, rather than the exception, that about two-thirds of the traffic travels in one direction on any rural highway during periods in which the highest total volumes occur. There are, however, no available data to show that any three-lane highway has ever accommodated more than 2,000 vehicles per hour in one direction; a number that can crowd into a single lane at points where the sight distance is too short for passing maneuvers to be performed safely.

Capacity

With a total of 1,500 passenger cars per hour, a three-lane highway with no restrictive sight distances will provide operating conditions comparable to those on two-lane and four-lane rural highways operating at their practical capacities.

The capacity of three-lane highways is very sensitive to restrictive sight distances.

The maximum practical capacity of a rural three-lane highway is therefore a total of 1,500 passenger cars per hour. This is 67 percent greater than the capacity of a two-lane highway and 75 percent of the capacity of two lanes for one direction of travel on a four-lane highway.

For less favorable sight-distance conditions, the difference between the practical capacities of a two- and a three-lane highway decreases and the difference between the practical capacities of a three- and a four-lane highway increases.

If sight distances sufficient to complete passing maneuvers safely are available only intermittently along a three-lane highway, the practical capacity of such a highway might be only slightly greater than the capacity of a two-lane road with the same alinement; or less than the capacity of a two-lane road with good alinement. **If a three-lane road is to accommodate traffic volumes substantially greater than those that can be accommodated by a good two-lane road, sight distances long enough to permit passing with safety must be almost continuous over the length of the road.**

Experience with three-lane highways

The past experience and practice of State highway departments should offer some index of the place of three-lane roads in our present highway system. Through the Bureau of Public Roads, the Highway Capacity Committee obtained data from the State highway departments showing the date of construction and traffic volumes during the life, or up to the present time, of most three-lane rural roads constructed in this country. Information was obtained from 27 States on more than 3,700 miles of three-lane roads, 7.8 percent of which had been converted to four or more lanes. It must be remembered, when interpreting the results of this study, that a lack of funds and the normal time required to plan, finance, and construct a facility, or other considerations, may have delayed the actual construction of a three-lane road (or the widening of a three-lane road to four lanes) several years after the actual need for the improvement became apparent, and that in some cases the inadequacy of an existing three-lane road was relieved by the construction of an alternate or parallel facility.

The average retirement age of three-lane highways converted to four lanes was 6.1 years, which is 1.6 years less than the average time that existing three-lane roads have been in service. The ages of three-lane roads when converted to four or more lanes varied from 15 years with an initial annual volume of less than 1,000 vehicles per day to less than 2 years for initial annual volumes exceeding 10,000 vehicles per day. The retired three-lane roads carried an average annual traffic volume of 7,029 vehicles per day as compared with an average maximum traffic volume to date of 4,996 vehicles per day for those still in existence. The traffic volumes on three-lane highways as initially constructed, on those still in existence, and on those that have been converted to wider highways, are shown in figure 17.

Prior to widening, 17 percent of the reported three-lane mileage carried annual average volumes of 10,000 and more vehicles per day. Only 5 percent of the existing three-lane mileage has carried annual volumes of 10,000 and more vehicles per day.

California and New Jersey each reported three-lane roads with exceedingly high annual volumes. On only one of these highways, however, has the peak hourly volume exceeded 2,000 vehicles. The following tabulation shows the peak hourly volumes on the three-lane roads in California carrying average annual volumes exceeding 10,000 vehicles per day.

Average daily traffic in year of highest annual volume:	*Peak hourly volume*
11,272	1,589
12,076	1,895
13,375	891
12,503	1,083
12,351	918
19,040	1,958

It is evident that these three-lane roads were taxed beyond their possible capacities during peak periods, and the peak hourly volumes were much lower than they would have been had the capacities of the roadways been greater.

Conclusions of studies

The more detailed results of the studies made by the Committee regarding three-lane roads may be obtained in mimeograph form from the Highway Research Board. The important conclusions of this study were:

1. At any point on a three-lane highway, relatively few vehicles travel in the center lane. The maximum number that can be in the center lane is about 300 per hour, regardless of the total traffic volume, when up to 70 percent of the total traffic is traveling in one direction.

2. Although there is a very marked drop in the average speed of traffic in the outside lanes with an increase in volume, there is no drop in the speeds of vehicles in the center lane.

3. As long as the hourly traffic volume traveling in one direction does not exceed 70 percent of the total traffic, the center lane will be used by vehicles traveling in both directions.

4. The average speed of all vehicles and the possible capacity of a three-lane road are slightly higher when the traffic is nearly evenly divided by direction than when two-thirds travel in one direction.

5. At places where sight distance is restricted, the use of the center lane for passing is dangerous; so, in effect, a three-lane highway will carry only two lanes of traffic at such points.

6. A three-lane highway having even one restricted sight distance cannot carry more vehicles per hour in one direction than the number that can crowd into one traffic lane—2,000 passenger cars per hour under ideal conditions.

7. Three-lane roads with an initial annual average volume of about 4,700 vehicles per day may be expected to give a useful service for approximately 6 years.

8. Three-lane roads with an initial annual average volume less than 3,300 vehicles per day may be expected to give satisfactory service for at least 13 years, and perhaps for the normal life of the pavement (15–20 years) providing traffic increases in the future at the same rate that it has in the past, excluding the war years.

Table 5.—*Roadway capacities for uninterrupted flows under ideal traffic and roadway conditions*

	Two-lane, two-way highway: total for both lanes [1]	Three-lane, two-way highway: total for all lanes [1]	Multi-lane highway: average per lane for direction of heavier flow [2]
	Passenger cars per hour	*Passenger cars per hour*	*Passenger cars per hour*
Basic capacity [3]	2,000	4,000	2,000
Practical capacity for urban conditions [4]	1,500	2,000	1,500
Practical capacity for rural conditions [5]	900	1,500	1,000

[1] Distribution by directions is not a factor.
[2] During periods of peak flow, traffic in one direction may be much heavier than in the other direction.
[3] Same as possible capacity for ideal conditions.
[4] Provides 35 to 40 m.p.h. operating speeds.
[5] Provides 45 to 50 m.p.h. operating speeds.

EFFECT OF FACTORS THAT REDUCE CAPACITIES

Table 5 summarizes the basic, possible, and practical hourly capacities of various types of highways under ideal traffic and roadway conditions when the flow is uninterrupted and sight distances are not restrictive.

It is seldom, however, that roadway and traffic conditions are ideal. For this reason, both possible and practical capacities for uninterrupted flow are usually lower than those shown in table 5. If surface condition be disregarded, and it is seldom a factor on a well-maintained highway with a high-type surface, the most important conditions which affect the capacity of the highway where the flow is not interrupted by cross traffic are: (1) lane width, (2) clearances to lateral obstructions, (3) shoulder width and condi-

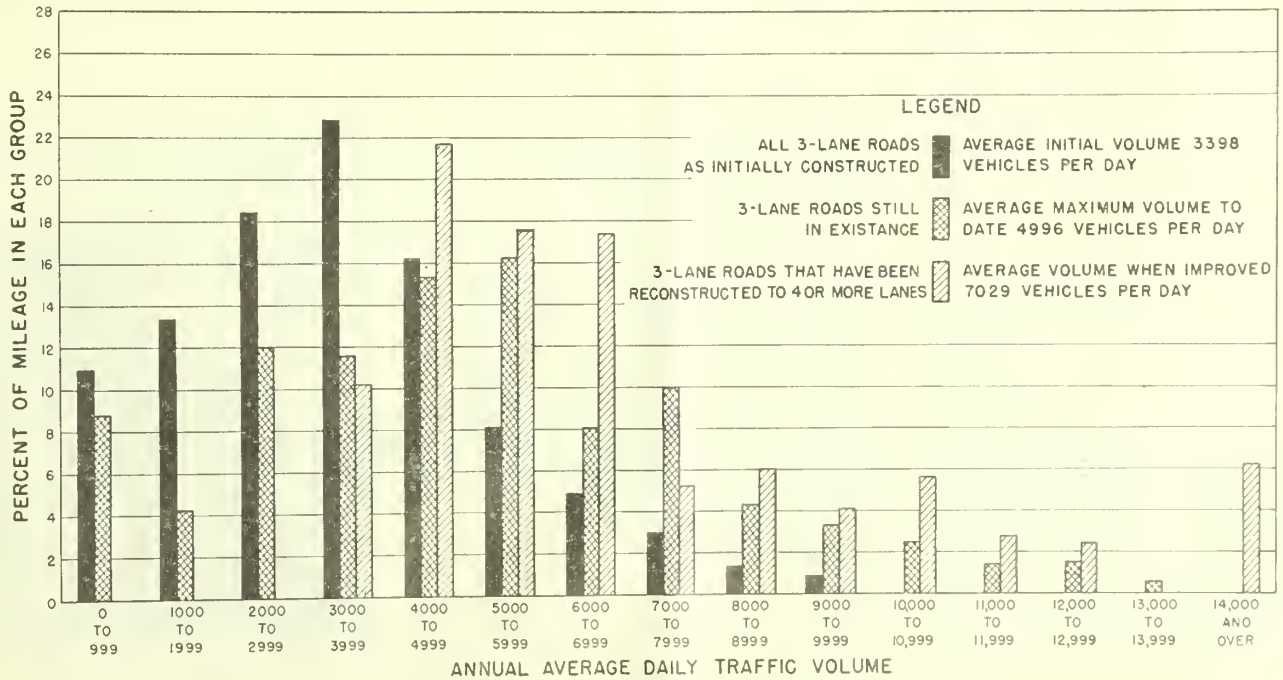


Figure 17.—Past experience with three-lane highways, based on all three-lane construction on State highways (totaling 3,740 miles).

Table 6.—*Effect of lane width on capacity* [1]

Lane width	Capacity expressed as a percentage of 12-foot lane capacity		
	Two-lane rural roads		Two lanes for one direction of travel on divided highways at practical capacities
	At possible capacities	At practical capacities	
Feet	*Percent*	*Percent*	*Percent*
12	100	100	100
11	88	86	97
10	81	77	91
9	76	70	81

[1] Effects of lane width on driver comfort, accident rates, etc., are not included in these relations.

Table 7.—*Effect of restricted lateral clearance on practical capacities* [1]

Clearance from pavement edge	Effective width of two 12-foot traffic lanes
Feet	*Feet*
6	24
4	23
2	21
0	18

[1] Effects of lateral clearance on driver comfort, accident rates, etc., are not included in these relations.

tion, (4) commercial vehicles, (5) location and design of interchange facilities, and (6) the profile and alinement, especially as related to sight distance and gradient.

Lane Width

Narrow lanes have a lower capacity than the 12-foot lanes which are at present considered necessary for heavy volumes of mixed traffic. On a two-lane road, a vehicle performing a passing maneuver must encroach on the lane normally used by traffic traveling in the opposite direction for a longer period if the lanes are narrow than if they are wide. On multilane roads, more vehicles straddle the lane lines when the lanes are narrow than when they are wide, thereby in effect occupying two lanes rather than one. Table 6 shows the capacity of lanes down to 9 feet in width on the basis of capacities for 12-foot lanes.

Narrow through-truss bridges seriously impair the capacity of many sections of highway.

This underpass offers no impediment to the free movement of traffic. Ample side clearance is assured even if a third lane is later added to the pavement.

Table 8.—*Combined effect of lane width and edge clearances on highway capacities* [1]

Clearance from pavement edge to obstruction	Capacity expressed as a percentage of the capacity of two 12-foot lanes with no restrictive lateral clearances							
	Obstruction on one side				Obstruction on both sides			
	12-foot lanes	11-foot lanes	10-foot lanes	9-foot lanes	12-foot lanes	11-foot lanes	10-foot lanes	9-foot lanes
POSSIBLE CAPACITY OF TWO-LANE HIGHWAY								
Feet								
6	100	88	81	76	100	88	81	76
4	97	85	79	74	94	83	76	71
2	93	81	75	70	85	75	69	65
0	88	77	71	67	76	67	62	58
PRACTICAL CAPACITY OF TWO-LANE HIGHWAY								
6	100	86	77	70	100	86	77	70
4	96	83	74	68	92	79	71	65
2	91	78	70	64	81	70	63	57
0	85	73	66	60	70	60	54	49
POSSIBLE AND PRACTICAL CAPACITIES OF TWO LANES FOR ONE DIRECTION OF TRAVEL ON DIVIDED HIGHWAYS								
6	100	97	91	81	100	97	91	81
4	99	96	90	80	98	95	89	79
2	97	94	88	79	94	91	86	76
0	90	87	82	73	81	79	74	66

[1] Effects of lane widths and lateral clearances on driver comfort, accident rates, etc., are not included in these relations.

The practical capacity of a two-lane rural road with lanes 9 feet wide, for example, is only 70 percent of the capacity of a similar road with 12-foot lanes.

Restrictive Lateral Clearances

Vertical obstructions such as retaining walls, light poles, and parked cars adjacent to the edge of a traffic lane reduce the effective width of that lane as shown by table 7. A 24-foot pavement with a bridge truss at the edge, for example, has the same effective width as an 18-foot pavement with 6-foot shoulders. In addition to their effect on capacity, lane width and lateral clearances also affect driving comfort, accident rates, etc., which the relationships shown by these tables do not include.

The combined effect on capacity of lane width and clearances from the pavement edge to obstruction are shown by table 8. Some judgment must obviously be exercised when applying these adjustments to highway sections where the lateral restrictions are not continuous along the entire length. One lateral restriction within a section of highway will cause a bottleneck and thereby directly affect the *possible* capacity of the entire sec-

Narrow shoulders, and shoulders that are poorly maintained, seriously impair the capacity of a highway.

A short-span bridge of inadequate roadway width.

tion. The *practical* capacity of the section, however, may be affected only slightly.

For example, one bridge 24 feet wide and 100 feet long on a highway with a 24-foot surface and a normal shoulder width of 8 feet would reduce the *possible* capacity of that entire section by 24 percent (from 100 to 76 percent, table 8). The *practical* capacity, however, would be influenced for only a short distance, thereby affecting the practical capacity of the entire section a lesser amount, in proportion to the relative lengths.

While no research data are available regarding the exact length of highway over which an individual obstruction within 6 feet of the pavement edge affects traffic operations, it seems reasonable, from the results of observations for other purposes, to assume that traffic will be affected to some extent for 9 seconds before reaching the lateral obstruction and that the net effect will be approximately the same as the full effect while traveling for 4½ seconds plus the distance over which the restrictive clearance continued. With traffic averaging 45 miles per hour the bridge in the above example would affect traffic over a distance of 397 feet (297+100). The practical capacity of the entire section of highway, if it were 1 mile long, would then be reduced by the narrow bridge from 100 percent, the practical capacity of a highway with 12-foot lanes and no lateral restrictions, to

$$\frac{(100\times 4{,}883)+(70\times 397)}{5{,}280}=93 \text{ percent of the}$$

capacity of a highway with 12-foot lanes and no lateral restrictions.

Table 8 shows the combined effect of lane width and lateral clearances for two-lane highways and for divided four-lane highways. For undivided four-lane highways, the lateral clearance on the left side of the lanes for travel in the one direction may be assumed to be equivalent to the distance from the left edge of these lanes to a vehicle centered in the adjacent lane used by traffic traveling in the opposite direction.

Likewise, when there are more than two lanes for the one direction of travel, interior lanes may be assumed to have the same capacity as lanes with lateral clearances equivalent to the distance between the edge of the lane and a vehicle centered in the adjacent lane. For example, if conditions are such that the practical capacity for one direction of travel on a four-lane divided highway with a wide median, 10-foot shoulders, and 12-foot lanes is 1,500 passenger cars per lane per hour, then, for similar conditions the practical capacity of the three 10-foot lanes for one direction of travel on a six-lane divided highway with a wide median and 10-foot shoulders would be:

For lanes 1 and 3 [3]	0.91×1,500×2=2,730
For lane 2 [4]	0.86×1,500 =1,290
Total	4,020

Shoulders

At no time are adequate shoulders more necessary on a highway than when the lanes are used to full capacity. Without a place of refuge outside the traffic lanes, one disabled vehicle can reduce the capacity of a highway by more than the capacity of one lane, especially if the lanes are less than 12 feet wide. The disabled vehicle blocks the lane occupied and, in addition, reduces the capacity of adjoining lanes whenever vehicles must merge into fewer lanes at speeds below 30 miles per hour. For example, the possible capacity of a traffic lane with vehicles moving at 20 miles per hour is only 87 percent of its capacity at 30 miles per hour. At 10 miles per hour a lane has only about 50 percent of its 30-mile-per-hour capacity (see fig. 3). A minor accident which causes a reduction in speed can, therefore, cause complete congestion on a facility operating near its possible capacity. For lanes less than 12 feet wide, shoulders treated with bituminous materials for a width of 4 feet or more increase the effective width of the adjacent traffic lanes by 1 foot.

[3] Factor of 0.91 from table 8, 10-foot lanes with no obstructions.

[4] Factor of 0.86 from table 8, 10-foot lane with clearances of 2 feet on either side to cars in adjoining lanes.

Bridge rails must be well removed from the edge of the pavement if they are to have no effect on the traffic capacity of a highway.

Consideration must be given to the needs of disabled vehicles, of which one may be expected for every eight to ten thousand vehicle-miles of travel, if all traffic is to be properly served.

Commercial Vehicles

Commercial vehicles reduce both the practical and possible capacities of a highway (see table 9) in terms of vehicles per hour because they occupy a greater road space and influence traffic over a larger area than do passenger cars. They also generally travel at lower speeds, especially on upgrades, thereby increasing the number of passing maneuvers that are necessary for other vehicles to maintain reasonable speeds. On multilane facilities with uninterrupted flow, one commercial vehicle (includes only those vehicles having

Table 9.—*Effect of commercial vehicles on practical capacities of multilane facilities*

Commercial vehicles	Capacity expressed as a percentage of passenger-car capacity on level terrain	
	Level terrain	Rolling terrain
Percent	*Percent*	*Percent*
None	100	100
10	91	77
20	83	63

The effect of trucks on capacity is greatest where alinement is poor and grades are long and steep.

dual tires on rear axle) has approximately the effect of two passenger cars in level terrain, and of four passenger cars in rolling terrain.

For example, approximately the same operating conditions will prevail on an expressway through rolling terrain when there are 1,500 passenger cars per lane per hour as when there are 115 trucks and 1,040 passenger cars per lane, a total of 1,155 vehicles.

In mountainous terrain the effect varies widely with the particular profile but, as an average, one commercial vehicle has approximately the same effect as eight passenger cars. The values of table 9 apply only to percentages of commercial vehicles within normal limits and do not include the effect of bus stops, etc. Care must be exercised in their application because the percentage of commercial vehicles during peak hours is generally considerably lower than the average percentage during all hours. On two-lane highways, the effect of commercial vehicles is about 25 percent greater than on multilane expressways.

Imperfect Alinement

The alinement and profile of a highway are important factors affecting its capacity at different operating speeds. In combination, they influence the sight distances along the highway or the length of roadway visible to the driver at any point on the roadway when the view is unobstructed by other traffic. To determine operating conditions on a highway, sight distance is divided into two categories: stopping sight distance and passing sight distance.

Stopping sight distance is the distance required by the driver of a vehicle, traveling at a given speed, to bring his vehicle to a stop after an object on the roadway becomes visible. Passing sight distance is the minimum sight distance that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably, without interfering with the speed of an oncoming vehicle should it come into view after the overtaking maneuver is started. Stopping sight distances are necessary continuously on all types of highways, whereas passing sight distances are necessary only on two-way roadways with two or three lanes.

Table 10.—*Percentage of total traffic and percentage of passed vehicles traveling at various speeds on two-lane highways where sight distances and oncoming traffic do not restrict passing opportunities*

Speed group (m. p. h.)	Average main rural highway		Highest-speed rural highways	
	All traffic	Passed vehicles	All traffic	Passed vehicles
	Percent	*Percent*	*Percent*	*Percent*
Over 50	11	1	39	5
40 to 49	57	15	43	40
30 to 39	30	54	17	40
Below 30	2	30	1	15
Total	100	100	100	100

Heavy grades, as a rule, have little effect on the speeds of passenger cars. It is the restrictive sight distances, which usually accompany steep grades, that create congestion at low volumes in mountainous terrain.

Where sight distances are inadequate on two- and three-lane highways, drivers are restricted in much the same manner as if the lane used for passing were filled with oncoming vehicles. The prudent driver must always assume the existence of an approaching vehicle just beyond the limit of his sight distance. The reduction in capacity caused by short sight distances can be obtained by using as a criterion the percentage of the total highway on which sight distances are insufficient to permit passing maneuvers to be performed safely.

The results of passing-practice studies conducted by the Bureau of Public Roads in cooperation with several State highway departments at locations where the alinement and profile provided unlimited opportunities for passing reflect the need for passing sight distances on two-lane rural highways. Table 10 shows the speed of the vehicles that were passed related to the speeds of all vehicles on the average main rural highway and on the highest-speed rural highways during periods of low traffic when passings could be made almost without interference from oncoming traffic.

Table 11.—*Effect of passing sight-distance restriction on practical capacities of two-lane highways when adequate stopping sight distances are always present* [1]

Percentage of total length of highway on which sight distance is restricted to less than 1,500 feet	Practical capacity, in passenger cars per hour—	
	For operating speed [2] of 45–50 miles per hour	For operating speed [2] of 50–55 miles per hour
0	900	600
20	860	560
40	800	500
60	720	420
80	620	300
100	500	160

[1] The data in this table apply to sections with 12-foot traffic lanes, shoulders adequate for parking disabled vehicles clear of the traffic lanes, and a continuous stopping sight distance corresponding to the design speed. Also, the sight distance on the restricted portions of the section must be uniformly distributed between the required stopping sight distance for the design speed and 1,500 feet.

[2] Average speed for drivers trying to travel at maximum safe speed.

Since these are the conditions that exist where alinement provides unlimited opportunities for passing, the greatest need on rural two-lane highways is for sight distances that will permit vehicles traveling under 50 miles per hour to be passed safely.

The results of the passing-practice studies show that for the most critical condition, when the passing vehicle first slows to the speed of the passed vehicle before accelerating to perform the passing maneuver, a sight distance to the road surface of 1,500 to 2,000 feet is required to pass a vehicle traveling between 45 and 50 miles per hour with the possibility of oncoming traffic traveling 70 miles per hour.

Photo by Portland Cement Association.

In mountainous terrain the effect of trucks on highway capacity can be minimized by the construction of added lanes on upgrades. This is not a three-lane road in the usual sense.

Passing sight distances within the range of 1,500 to 2,000 feet are, therefore, those most widely needed at frequent intervals on rural highways.

Where sight distances within the range of 1,500 to 2,000 feet are not continuously available throughout the length of a two-lane highway, the percentage of the total length of highway with a 1,500-foot sight distance can be used as a criterion of the highway's practical capacity. Table 11 shows the reduction in capacity caused by sight-distance restrictions when operating speeds of 45 to 50 and of 50 to 55 miles per hour are desired.

Figure 18 shows in more detail the effect of passing sight-distance restrictions on the capacity of two-lane highways for various operating speeds.

Grades

Grades affect the capacity of a highway in three ways, as follows:

1. Vehicle braking distance is less on upgrades and greater on down grades than on the level, thereby permitting shorter spacings between vehicles that are climbing grades, and requiring longer spacings between vehicles going down grades, in order to maintain a safe headway.

2. The presence of a grade generally causes a restriction in the sight distance, thereby affecting the percentage of highway on which passing maneuvers can be performed safely.

3. Commercial vehicles with their normal loads travel at slower speeds up grades than on the level, especially if the grade is long and steep. This is also true to some extent for passenger cars Most passenger cars, however, can negotiate long 6- and 7-percent grades at speeds above 30 miles per hour. The effect that grades up to 7 percent have on capacity as related to the performance of passenger cars is therefore generally negligible.

In the discussion of commercial vehicles, it was stated that one commercial vehicle on a multilane highway has approximately the same effect on capacity as two passenger cars in level terrain and as four passenger cars in rolling terrain. It was also stated that their effect on capacity for two-lane highways is 25 percent greater than on multilane highways. The figures refer to general conditions over the entire length of a highway. In considering the effect of an individual grade, the length and rate of grade are important factors.

The relationships between speed of trucks at the bottom of a hill, percentage of grade, and distance upgrade are shown in figure 19 for light-powered motortrucks or combination units and in figure 20 for medium-powered motortrucks or combination units in operation in 1941.

Table 12.—*Distance from bottom of grade at which speed of trucks is reduced to 30 miles per hour* [1]

Grade	Distance from bottom of grade	Vertical climb from bottom of grade
LIGHT-POWERED TRUCKS WITH GROSS LOADS OF 30,000 POUNDS		
Percent	*Feet*	*Feet*
2	2,000	40
3	1,090	33
4	760	30
5	570	29
6	470	28
7	400	28
8	325	26
MEDIUM-POWERED TRUCKS WITH GROSS LOADS OF 40,000 POUNDS		
2	1,780	36
3	1,035	31
4	740	30
5	550	28
6	450	27
7	390	27
8	320	26

[1] Assuming an approach speed of 40 m.p.h. Bad alinement, weak or narrow bridges, or other hazardous conditions at the bottom of the hill would make this approach speed unsafe.

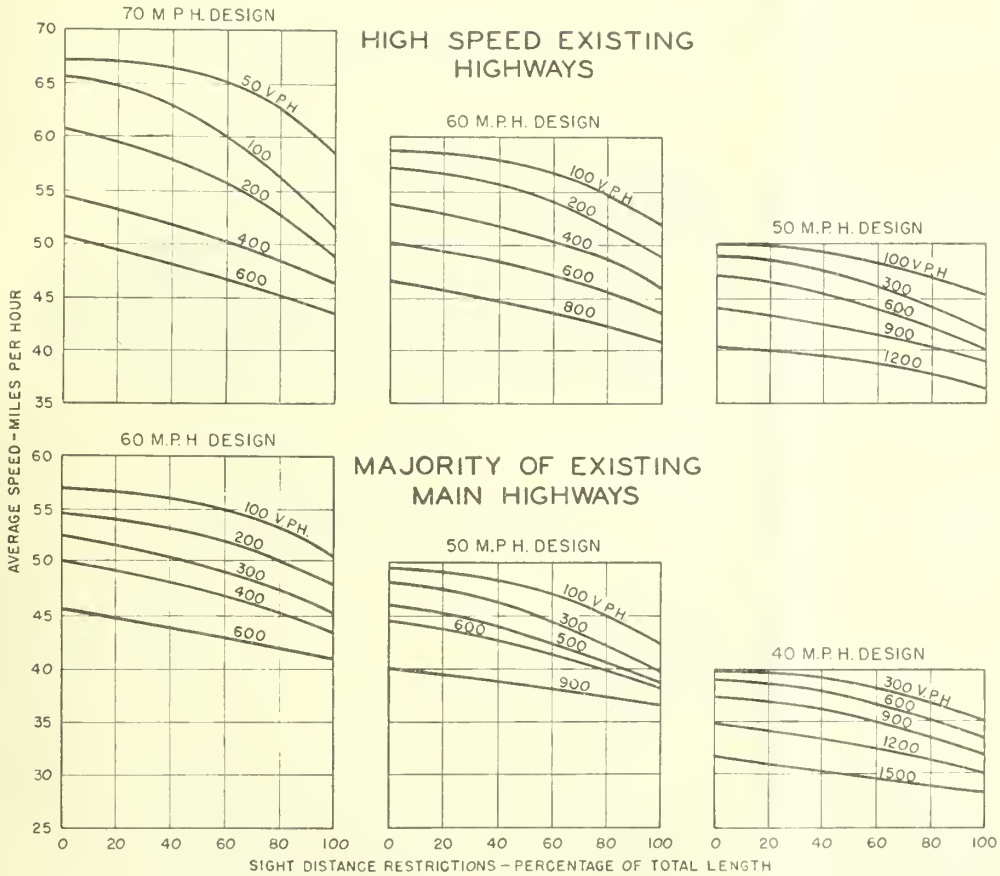


Figure 18.—Possible average speed at different traffic volumes, for drivers trying to travel at the design speed, when the sight distances on various portions of the highway are less than the passing sight distance.

The light-powered vehicles had engines which, on an average, would develop 93 brake horsepower; the medium-powered vehicles had engines which, on an average, would develop 106 brake horsepower. Heavy-powered vehicles in operation in 1941 had engines which would develop, on an average, 115 brake horsepower. At the present time, light-powered vehicles will develop nearly the same brake horsepower as those considered medium-powered in 1941.

From these data, for the power and gross weights of vehicles represented, it is possible to determine how far a vehicle, starting its climb from any speed between 9 and 41 miles per hour, can travel up various grades or combinations of grades before the maximum sustained speed reaches any stated value. The solid curves in figures 19 and 20 indicate the performance that may be expected when the beginning speed is above the possible sustained or crawl speed. The broken lines, starting at 9 miles per hour, show what performance may be expected when the hill is approached at crawl speed.

Table 12, for example, shows the distance that light-powered trucks with gross loads of 30,000 pounds, and medium-powered trucks with gross loads of 40,000 pounds, can go up various grades before their speeds are lowered to 30 miles per hour, assuming that they enter the grade at 40 miles per hour. It will be noted that the length of grade that reduces the speed to 30 miles per hour is approximately the same for the light-powered trucks with gross loads of 30,000 pounds as for the

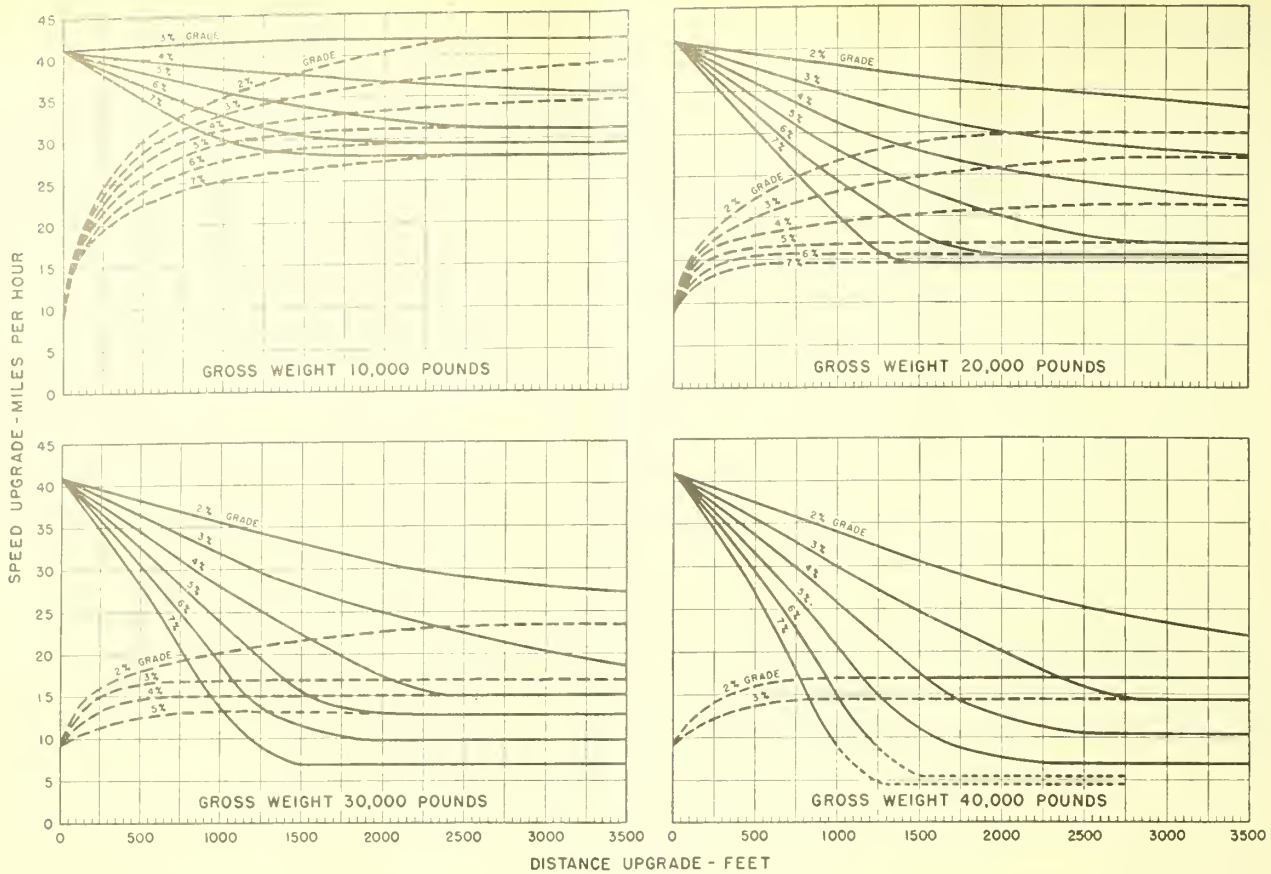


Figure 19.—Effect of length of grade on the speed of light-powered trucks and combinations.

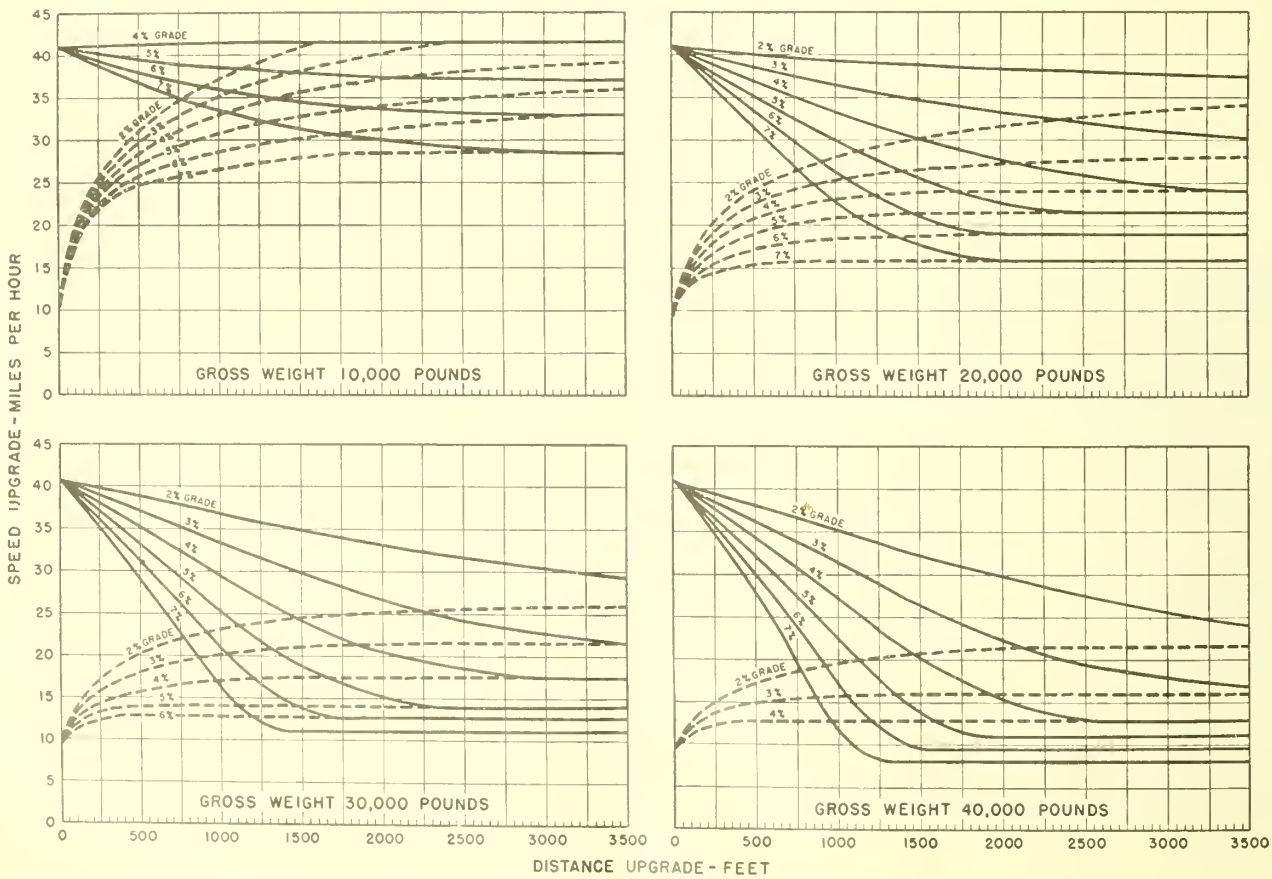


Figure 20.—Effect of length of grade on the speed of medium-powered trucks and combinations.

Table 13.—*Effect of commercial vehicles and grades on the capacity of two-lane highways with uninterrupted traffic flow when the grade does not cause a restriction in the passing sight distance*

Length of grade	Equivalent of one dual-tired commercial vehicle, in terms of passenger cars, on a grade averaging—				
	3 percent	4 percent	5 percent	6 percent	7 percent
Miles					
0.1	3.9	4.1	4.2	4.2	4.4
0.2	4.1	4.3	4.5	4.7	5.1
0.4	4.3	4.6	4.9	5.3	5.5
0.6	4.4	4.8	5.2	5.8	6.5
0.8	4.6	5.1	5.7	6.4	7.1
1.0	4.6	5.3	6.0	6.7	7.4
1.5	4.8	5.6	6.3	7.0	7.7
2.0	5.0	5.8	6.5	7.2	8.0
3.0	5.0	6.0	6.6	7.3	8.2
4.0	5.1	6.0	6.7	7.4	8.3
5.0	5.1	6.0	6.7	7.6	8.3
6.0	5.1	6.0	6.8	7.6	8.3

medium-powered trucks with gross loads of 40,000 pounds. Grades longer than those shown in table 12 would, therefore, have an effect on the possible capacity of a highway because they reduce the speed of trucks that occur with considerable frequency to speeds below 30 miles per hour, the optimum speed for maximum capacity.

If, for the time being, we disregard the effect that grades have in causing sight-distance restrictions, the effect of one commercial vehicle on highway capacity, in terms of passenger cars, is as shown in table 13. There would obviously be an additional effect due to the restrictive sight distances on the grade which would prevent passing maneuvers from being performed as readily as on a level section. The effect due to restrictive sight distances, as shown in table 11, must therefore be added to the effect shown in table 13. To illustrate the total effect that one commercial vehicle has on capacity in terms of passenger cars, including the effect of imperfect alinement, table 14 has been prepared assuming that the percentage of restrictive sight distance increases from 30 percent for a 3-percent grade to 70 percent on a 7-percent grade. For specific conditions, these percentages should be modified to correspond with actual conditions.

The data shown in table 14 can be applied to a number of problems: For example, table 15 shows the relation of length and rate of grade and of traffic volume to the type of facility required in order to provide operating conditions equivalent to conditions on a level stretch with 800 vehicles per hour, including 10 percent commercial vehicles.

Table 14.—*Effect of commercial vehicles and grades on the capacity of two-lane highways with uninterrupted traffic flow, with passing sight distance restricted for 1,500 feet ahead of the crest and a typical alinement elsewhere along the grade*

	Equivalent of one dual-tired commercial vehicle, in terms of passenger cars, on a grade averaging—				
	3 percent	4 percent	5 percent	6 percent	7 percent
Percentage of highway with restricted sight distance (assumed as typical)	30	40	50	60	70
Capacity of highway with restricted sight distance, as a percentage of unrestricted capacity	87	81	75	69	62
Length of grade, in miles:					
0.1	5.8	7.1	8.6	10.2	12.5
0.2	6.0	7.4	9.0	10.9	13.6
0.4	6.3	7.8	9.6	11.6	14.6
0.6	6.5	8.2	10.1	12.5	16.1
0.8	6.6	8.5	10.6	13.0	16.8
1.0	6.7	8.7	11.0	13.5	17.4
1.5	6.9	9.0	11.5	14.1	18.2
2.0	7.0	9.3	11.9	14.5	18.4
3.0	7.1	9.4	12.0	14.7	19.0
4.0	7.1	9.4	12.1	14.7	19.1
5.0	7.2	9.5	12.1	14.8	19.1
6.0	7.3	9.6	12.1	14.9	19.1

Table 15.—*Effect of grades on type of highway facility required when the traffic includes 10 percent commercial vehicles with dual rear tires*

Gradient	Traffic volume	Length of grade	Type of facility required to provide satisfactory operating conditions for passenger cars [1] [2]		
			Two-lane highway	Two-lane highway with truck lane on uphill side	Four-lane highway
Percent	*Vehicles per hour*				
0	800 or less	Any length	×	--	--
	Over 800 [3]	Any length	--	--	×
3	550 or less	Any length	×	--	--
	551 to 800	1,100 feet or less	×	--	--
		Over 1,100 feet	--	×	--
	Over 800 [3]	Any length	--	--	×
4	500 or less	Any length	×	--	--
	501 to 800	800 feet or less	×	--	--
		Over 800 feet	--	×	--
	Over 800 [3]	Any length	--	--	×
5	400 or less	Any length	×	--	--
	401 to 800	600 feet or less	×	--	--
		Over 600 feet	--	×	--
	Over 800 [3]	Any length	--	--	×
6	350 or less	Any length	×	--	--
	351 to 400	4,000 feet or less	×	--	--
		Over 4,000 feet	--	×	--
	401 to 800	Under 500 feet	×	--	--
		500 to 4,000 feet	--	×	--
		Over 4,000 feet	--	--	×
	Over 800 [3]	Any length	--	--	×
7	300 or less	Any length	×	--	--
	301 to 350	4,000 feet or less	×	--	--
		Over 4,000 feet	--	×	--
	351 to 800	Under 500 feet	×	--	--
		500 to 4,000 feet	--	×	--
		Over 4,000 feet	--	--	×
	Over 800 [3]	Any length	--	--	×

[1] Satisfactory operating conditions for passenger cars are here considered as the equivalent of a capacity of 800 vehicles per hour on level sections of a two-lane highway.

[2] This table is based on the assumptions that a three-lane highway on which traffic in both directions has an equal right in the center lane has no place in modern highway design. Also, when the hourly traffic volume used for design purposes (30th highest hourly volume in the year) exceeds 800 vehicles, a four-lane divided highway is required for any condition of gradient or alinement. If the sight distance is restricted to less than 1,500 feet on portions of the highway other than the grade under consideration, an added truck lane or a four-lane highway may be needed for traffic volumes lower than those shown in the table.

[3] If volume is sufficiently high, a truck lane may be necessary on the uphill side of a four-lane highway, or a multilane highway of more than four lanes may be required.

CAPACITIES EXPRESSED AS AVERAGE ANNUAL DAILY TRAFFIC VOLUMES

The relation between peak hourly rates of flow and annual average daily traffic volumes is the subject of part VIII of this report. There, a method is outlined for converting peak hourly volumes into average daily volumes. It is shown that if the average daily traffic is known, the hourly volume during peak periods can be estimated by applying certain known factors. Conversely, the hourly capacity for a highway may be converted to an average annual daily volume by applying the reciprocal of the factor. For most rural highways the factor for this purpose is between 15 and 16 percent, with a typical value of 15.6 percent. Thus, for a two-lane rural highway with ideal conditions, the average daily volume which will correspond to the practical capacity of 900 vehicles per hour is $900 \div 0.156 = 5{,}750$ vehicles per day. This is an average value and consequently will differ according to locality. As a matter of interest, table 16 has been prepared for the purpose of showing average annual daily traffic volumes corresponding to the practical capacities of various types of highways, using the factor 15.6 percent.

It should be obvious that volumes of the magnitude shown in this table are seldom achieved without exceeding the practical capacity, because the high design standards upon which they are based can be economically justified in rare cases only. Particularly is this true for two-lane roads, the capacities of which are very sensitive to restrictive sight distances.

In rolling terrain the alinement may be such as to cause a reduction of 50 percent or more in the average daily volumes for two-lane roads as shown in table 16. For any specific highway, an evaluation of the several factors affecting capacity must precede the computation of a reasonable value for the annual average daily traffic volume.

Table 16.—*Average annual daily traffic volumes corresponding to the practical capacities of different types of highways, as based on the Nation-wide average relationship between the thirtieth highest hourly volume and the average annual daily traffic volume* [1]

Type of traffic		Average annual daily traffic volume on—					
		Two-lane rural roads		Four-lane rural roads [2]		Four-lane urban expressways [2]	
Percentage of passenger cars	Percentage of commercial vehicles	In level terrain	In rolling terrain	In level terrain	In rolling terrain	In level terrain	In rolling terrain
100	0	5,750	5,750	19,250	19,250	37,500	37,500
90	10	5,200	4,450	17,500	14,800	34,000	29,000
80	20	4,800	3,600	16,050	12,000	31,000	23,500

[1] Except for the presence of commercial vehicles, roadway and traffic conditions are assumed to approach the ideal, including 12-foot traffic lanes, tangent alinement, and uninterrupted flow.

[2] Assuming two-thirds of traffic in heavier direction during peak hour.

APPLICATION OF CAPACITY INFORMATION TO SPECIFIC PROBLEMS

The following examples illustrate correct applications of the data contained in this chapter to specific conditions. In each example the problem is stated first, and the solution is then worked out in three successive steps: First, the capacity under ideal conditions is determined; next, the adjustment factors for existing conditions are obtained from the tables; and finally, these factors are applied to the ideal-condition capacities.

Example 1

Problem

What is the possible capacity of one tube of the Holland Tunnel during periods when 10 percent of the traffic is heavy commercial vehicles? The tube has a 20-foot roadway between curbs, a 1-foot clearance to the vertical walls on each side, and a 4-percent grade.

Solution

For ideal conditions: possible capacity = 2×2,000 = 4,000 passenger cars per hour.

Adjustments:

	Factor
Surface width and lateral clearances [5]	0.80
Commercial vehicles [6]	.77
Combined factor	0.616

Application of factor: possible capacity = 4,000×0.616 = 2,464 vehicles per hour.

[5] Factor obtained from table 8.

[6] Factor obtained from table 9.

Example 2

Problem

What are the possible and practical hourly capacities of a two-lane rural highway with a 20-foot surface and with frequent obstructions within 4 feet of both pavement edges, located in rolling terrain where 10 percent of the peak-hour traffic is commercial vehicles and the sight distance is restricted to less than 1,500 feet over 60 percent of its length?

Solution

For ideal conditions:

Possible capacity = 2,000 passenger cars per hour.

Practical capacity = 900 passenger cars per hour to provide operating speed of 45 to 50 m.p.h.

Adjustments:

	Possible capacity factor	*Practical capacity factor*
Surface width and lateral clearance [7]	0.76	0.71
Commercial vehicles [8]	.67	.67
Alinement [9]	1.00	.80
Combined factor (product of individual factors)	0.509	0.381

Application of factors:

Possible capacity = 2,000×0.509 = 1,018 vehicles per hour.

Practical capacity = 900×0.381 = 343 vehicles per hour.

[7] Factors obtained from table 8.

[8] Factors obtained from table 9, further corrected for increased effect of commercial vehicles on two-lane roads.

[9] Factors obtained from table 11 (practical capacity factor = 720÷900).

Example 3

Problem

What are the possible and practical capacities of the upper roadway of the San Francisco-Oakland Bay Bridge, which carries passenger cars only, and has three lanes each 9.5 feet wide for traffic traveling in each direction? The roadway is not divided and has high vertical curbs.

Solution

For ideal conditions:

Possible capacity = 2,000 passenger cars per lane.

Practical capacity for operating speed of 35 to 40 m. p. h. = 1,500 passenger cars per lane.

Adjustments (necessary only for the lane widths and lateral clearances):

	Clearance in feet		
	Right	*Left*	*Factor* [10]
Lane 1	0	1.5	0.74
Lane 2	1.5	1.5	.78
Lane 3	1.5	1.5	.78

Application of factors:

	Possible capacity	*Practical capacity*
Lane 1	1,480	1,110
Lane 2	1,560	1,170
Lane 3	1,560	1,170
Total, vehicles per hour	4,600	3,450

[10] To obtain factor for lane 1, interpolate in table 8 for an average factor between values of zero feet on both sides and clearance of 1½ feet on both sides. Then interpolate between these average factors for 9- and 10-foot lanes.

Estimated Motor-Vehicle Registrations, 1949

Motor-vehicle registrations and the volume of highway travel in the United States continued their upward climb through the first half of 1949 and will surpass last year's record levels before the end of the year, according to preliminary estimates prepared by the Bureau of Public Roads.

Registration reports from State authorities for the first 6 months of 1949 indicate that approximately 43,298,000 private and commercial motor vehicles will be registered during the calendar year, an increase over 1948 registrations of 2,230,000 automobiles, or 6.7 percent, and 446,000 trucks and busses, or 6.1 percent. A tabulation of the estimates, by class of vehicle and by State, appears on the following page.

These estimates do not include vehicles owned by the Federal, State, county, and municipal governments. The number of publicly owned motor vehicles registered in 1948 was 529,062.

The estimated number of automobiles, busses, and trucks that will be registered in 1949 represents a 10-year growth in the motor-vehicle population of 41.4 percent, compared with 15.5 percent for the previous decade of 1930 through 1939. The greatest increase has occurred in trucks and busses, which have risen 74.4 percent over 1939 and 126.8 percent over 1929, while automobiles have increased 35.8 percent since 1939 and 53.9 since 1929.

It now appears, however, that 1949 will be the first year since 1941 in which the percentage increase over the previous year's registrations will be greater for automobiles than for trucks and busses. This is largely the result of the fact that the production of trucks increased more rapidly than that of passenger cars during the immediate postwar years. In 1949 it is expected that automobile production will be the greatest in history, but truck production will be less than for 1948.

The greater increase in automobile registrations resulted, of course, in a slight decrease in the proportion of trucks and busses to the total number of vehicles registered. Contrary to the long-term trend in which trucks and busses have risen from 13.0 percent of total registrations in 1929 to 14.6 percent in 1939 and 18.1 in 1948, they declined slightly to 18.0 percent in 1949.

The relative increases in motor-vehicle registrations during the last two decades have been accompanied by even greater increases in the consumption of motor fuel. With a tentatively estimated 31,400,000,000 gallons to be consumed on the highways by private and commercial motor vehicles in 1949, the increase from 1939 is 52.1 percent and from 1929 is 122.1 percent. The 1949 estimate represents a gain of 5.0 percent over 1948, compared with an increase in 1948 of 7.9 percent over 1947.

The increases in the number of vehicles registered and motor fuel consumed are reflected in the most recent data on rural traffic volumes. It is estimated that there were 2,167,200,000 miles of travel on rural roads in July, the latest month for which figures are available. This is an increase of 7.5 percent over the volume of travel on rural roads in July of last year, and is 25.2 percent more than the volume reported for July 1941.

UNITED STATES DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

PRELIMINARY ESTIMATE OF 1949 MOTOR-VEHICLE REGISTRATIONS 1/

(DOES NOT INCLUDE PUBLICLY-OWNED VEHICLES)

AUGUST 1949

STATE	AUTOMOBILES			TRUCKS AND BUSSES			TOTAL		
	REGISTERED 1948 2/	ESTIMATED 1949	PERCENT INCREASE 1949/1948	REGISTERED 1948 2/	ESTIMATED 1949	PERCENT INCREASE 1949/1948	REGISTERED 1948 2/	ESTIMATED 1949	PERCENT INCREASE 1949/1948
ALABAMA	391,704	425,000	8.5	141,370	151,000	6.8	533,074	576,000	8.1
ARIZONA	161,547	175,000	8.3	49,450	53,000	7.2	210,997	228,000	8.1
ARKANSAS	256,527	270,000	5.3	125,906	135,000	7.2	382,433	405,000	5.9
CALIFORNIA	3,194,226	3,350,000	4.9	554,081	590,000	6.5	3,748,307	3,940,000	5.1
COLORADO	354,748	375,000	5.7	107,588	115,000	6.9	462,336	490,000	6.0
CONNECTICUT	536,867	568,000	5.8	88,612	92,000	3.8	625,479	660,000	5.5
DELAWARE	68,240	72,000	5.5	18,816	20,000	6.3	87,056	92,000	5.7
FLORIDA	616,432	670,000	8.7	154,503	165,000	6.8	770,935	835,000	8.3
GEORGIA	540,984	585,000	8.1	166,833	175,000	4.9	707,817	760,000	7.4
IDAHO	156,979	168,000	7.0	60,590	65,000	7.3	217,569	233,000	7.1
ILLINOIS	1,899,305	2,050,000	7.9	305,761	320,000	4.7	2,205,066	2,370,000	7.5
INDIANA	1,059,447	1,115,000	5.2	236,010	250,000	5.9	1,295,457	1,365,000	5.4
IOWA	734,281	785,000	6.9	152,917	165,000	7.9	887,198	950,000	7.1
KANSAS	570,705	605,000	6.0	179,060	192,000	7.2	749,765	797,000	6.3
KENTUCKY	468,901	505,000	7.7	139,921	150,000	7.2	608,822	655,000	7.6
LOUISIANA	390,663	430,000	10.1	121,926	132,000	8.3	512,589	562,000	9.6
MAINE	187,132	192,000	2.6	61,585	62,000	0.7	248,717	254,000	2.1
MARYLAND	469,500	495,000	5.4	99,582	102,000	2.4	569,082	597,000	4.9
MASSACHUSETTS	943,329	995,000	5.5	155,464	159,000	2.3	1,098,793	1,154,000	5.0
MICHIGAN	1,741,613	1,870,000	7.4	237,716	250,000	5.2	1,979,329	2,120,000	7.1
MINNESOTA	798,144	860,000	7.7	170,334	185,000	8.6	968,478	1,045,000	7.9
MISSISSIPPI	263,239	285,000	8.3	127,735	140,000	9.6	390,974	425,000	8.7
MISSOURI	876,882	915,000	4.3	227,248	240,000	5.6	1,104,130	1,155,000	4.6
MONTANA	145,800	155,000	6.3	69,993	76,000	8.6	215,793	231,000	7.0
NEBRASKA	394,306	410,000	4.0	109,509	115,000	5.0	503,815	525,000	4.2
NEVADA	47,328	50,000	5.6	13,671	14,000	2.4	60,999	64,000	4.9
NEW HAMPSHIRE	119,611	125,000	4.5	37,364	39,000	4.4	156,975	164,000	4.5
NEW JERSEY	1,112,470	1,195,000	7.4	205,249	215,000	4.8	1,317,719	1,410,000	7.0
NEW MEXICO	126,458	140,000	10.7	47,160	51,000	8.1	173,618	191,000	10.0
NEW YORK	2,721,884	2,920,000	7.3	434,007	450,000	3.7	3,155,891	3,370,000	6.8
NORTH CAROLINA	659,725	702,000	6.4	171,981	183,000	6.4	831,706	885,000	6.4
NORTH DAKOTA	162,796	173,000	6.3	70,748	78,000	10.3	233,544	251,000	7.5
OHIO	2,096,623	2,230,000	6.4	299,925	320,000	6.7	2,396,548	2,550,000	6.4
OKLAHOMA	508,318	545,000	7.2	164,922	180,000	9.1	673,240	725,000	7.7
OREGON	448,545	485,000	8.1	116,232	128,000	10.1	564,777	613,000	8.5
PENNSYLVANIA	2,131,326	2,290,000	7.4	411,306	430,000	4.5	2,542,632	2,720,000	7.0
RHODE ISLAND	190,452	201,000	5.5	31,597	33,000	4.4	222,049	234,000	5.4
SOUTH CAROLINA	381,152	408,000	7.0	95,390	103,000	8.0	476,542	511,000	7.2
SOUTH DAKOTA	185,953	199,000	7.0	60,468	68,000	12.5	246,421	267,000	8.4
TENNESSEE	518,604	555,000	7.0	141,736	151,000	6.5	660,340	706,000	6.9
TEXAS	1,774,498	1,910,000	7.6	497,472	525,000	5.5	2,271,970	2,435,000	7.2
UTAH	164,538	177,000	7.6	40,575	43,000	6.0	205,113	220,000	7.3
VERMONT	95,407	98,000	2.7	15,266	16,000	4.8	110,673	114,000	3.0
VIRGINIA	610,301	665,000	9.0	147,996	155,000	4.7	758,297	820,000	8.1
WASHINGTON	623,913	670,000	7.4	147,815	159,000	7.6	771,728	829,000	7.4
WEST VIRGINIA	302,356	317,000	4.8	97,135	102,000	5.0	399,491	419,000	4.9
WISCONSIN	829,100	865,000	4.3	199,050	210,000	5.5	1,028,150	1,075,000	4.6
WYOMING	82,431	90,000	9.2	30,389	33,000	8.6	112,820	123,000	9.0
DISTRICT OF COLUMBIA	146,164	156,000	6.7	20,846	22,000	5.5	167,010	178,000	6.6
TOTAL	33,261,454	35,491,000	6.7	7,360,810	7,807,000	6.1	40,622,264	43,298,000	6.6

1/ THE ESTIMATES ARE FOR THE CALENDAR YEAR 1949 AND ARE BASED ON INFORMATION SUPPLIED BY STATE AUTHORITIES. PUBLICLY-OWNED VEHICLES ARE NOT INCLUDED IN THIS TABLE. THE NUMBER OF VEHICLES OWNED BY FEDERAL, STATE, AND LOCAL GOVERNMENTS IN 1948 WERE AS FOLLOWS:

AUTOMOBILES	136,989
BUSSES	64,123
TRUCKS	327,950
TOTAL	529,062

2/ REGISTRATION PERIODS ENDING NOT EARLIER THAN NOVEMBER 30 AND NOT LATER THAN JANUARY 31 ARE TREATED AS CALENDAR-YEAR PERIODS. IN THOSE STATES WHERE THE REGISTRATION PERIOD IS DEFINITELY REMOVED FROM THE CALENDAR YEAR, REGISTRATION FIGURES ARE GIVEN FOR THE TWELVE CONSECUTIVE MONTHS OF THE CALENDAR YEAR. FOR DETAILS OF 1948 REGISTRATIONS, SEE TABLE MV-1, 1948.

U. S. GOVERNMENT PRINTING OFFICE: 1949

Highway Capacity:

Practical Applications of Research

BY THE COMMITTEE ON HIGHWAY CAPACITY
DEPARTMENT OF TRAFFIC AND OPERATIONS
HIGHWAY RESEARCH BOARD

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Part V.—Signalized Intersections

INTRODUCTION

In part IV it was shown that the basic capacity of any highway facility is seldom achieved because of the effect of a large number of variable factors that tend to act as retardants to free vehicular movements. These variable factors were enumerated and measures of their effect have, in some cases, already been determined. One of the more important elements limiting the capacity of any facility, especially that of city streets, is the intersection at grade. Because of the many variations in the lay-out of intersections and the multiplicity of impediments which are usually inherent in city traffic, the capacity of intersections is here treated as a separate subject.

The Highway Capacity Committee found that very little material had been published on this subject, so it was necessary that some research be performed before satisfactory data could be assembled. In addition to the data collected by committee members, a large amount of material was furnished by most of the State highway departments, and by officials of many cities, as a result of solicitation by the Bureau of Public Roads. Some cities, including Chicago, Philadelphia, Milwaukee, and Washington, assigned several men to the task of obtaining information on a scale never before attempted. The analysis represents the conditions reported for hundreds of intersections and many months of tedious work by the Bureau of Public Roads. There will, of course, be additions and refinements to this report which may be issued as supplements as further analyses are made or as more data for specific conditions become available. There is little possibility, however, that additional data obtained within any reasonable length of time will materially change the present results.

There are certain factors that influence intersection capacities for which there are no available data. Particularly perplexing among these is the extent to which environmental and operating characteristics, which might vary widely between different localities, influence the number of vehicles that can pass through an intersection in a period of time. Local traffic regulations, the degree of enforcement, and the education and training of drivers are among the elements that comprise these environmental conditions which cannot be evaluated from the material at hand. Their average effect, however, is included.

A city street loaded to its possible capacity during rush hour.

The number of usable observations for some of the conditions covered is less than would be desirable, but the results have in all cases been based on far more information than has heretofore been available.

UNITS FOR EXPRESSING SIGNALIZED INTERSECTION CAPACITY

Current practice in expressing intersection capacity varies widely among individuals and this accounts in part for some of the controversies that have been known to develop in the past over the numbers of vehicles that can clear an intersection in an hour. It is important that the reader have an understanding of the units of measure used in this report, for otherwise the results are subject to misinterpretation.

Basically, an intersection consists of an intersection area and a number of legs or roadways on which vehicles approach and leave the intersection area. The traffic signals limit the number of approaches on which vehicles may move at one time, whereas the use of exit lanes is generally not restricted by the traffic signal.

At some locations, where all legs are not of the same width or where traffic backs up from an adjacent intersection, the capacity of the intersection may be dependent upon the capacity of the exit roadways. Generally, however, the capacity of the approaches controls the capacity of the intersection. It is also seldom that all approaches are burdened to their full capacities simultaneously. The maximum total volume from all approaches might conceivably occur when no one of the approaches was congested. It is appropriate, therefore, that intersection capacity be thought of in terms of the capacity of each of the approaches.

The number of vehicles that can enter an intersection is dependent upon a large number

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A downtown street in a large city during an off-peak period. The traffic load is near the practical capacity of the intersection.

of factors, some of which are variable while others are fixed or semifixed. Mere numbers of vehicles are of little value unless they are accompanied by sufficient information to permit their evaluation in consideration of these factors. As a minimum requirement, a unit of time and a unit of roadway width are essential. These two units, together with the number of vehicles, can be incorporated in an expression of the rate of flow relative to time and street width.

That portion of time occupied by the red or STOP signal indication has no utility value insofar as the traffic which it holds at a standstill is concerned. This fixed portion of the time is lost to that traffic, there being a zero rate of flow. Hence, only the time during which the signal is green (the go signal) is used in calculating rates of flow, and the hour of green is the unit in which it is expressed.

By applying the fraction of the total time that the signal is green for a particular movement to a known rate of flow in terms of vehicles per hour of green, the number of vehicles that can enter the intersection from that approach during 1 hour of total elapsed time can be readily calculated. In applying the capacity values given later in the report, the reader is cautioned against adding the amber time or any portion thereof to the green time in calculating the percentage of the total signal cycle during which traffic on one approach leg of the intersection is free to move. This would produce erroneous results because vehicles that entered during the amber period have been combined with those that entered during the green period to obtain the hourly rates shown in this report.

The basic unit of width used in expressing the capacity of roadways is the traffic lane. When the analysis of the intersection data was begun, it was also believed that the number of lanes would be a most important controlling factor and that streets of certain widths which would provide for an even number of 9- to 11-foot lanes would be more efficient than streets that were somewhat narrower or wider. It was believed, for example, that a street with a clear width between curbs of 20 feet for one lane in each direction or a clear width of 40 feet for two lanes in each direction would be much more efficient per foot of width than, for example, a street 27 feet wide for two-directional flow. Trial analysis of the data revealed, however, that intersection capacity varies in almost direct ratio with the width of the approach, measured from the curb line, and that the results for comparable traffic conditions were more consistent when based on the capacity per foot of width than when based on the capacity per traffic lane.

For example, a 64-foot street had the same capacity per foot of width if it was striped for either eight or six lanes, whereas the capacity per lane was greater for the six wider lanes than for the eight narrower lanes. With the wider lanes, the vehicles in the same lane travel closer together and fewer vehicles straddle the lane lines than they do when the lanes are narrow.

The basic unit "vehicles per 1 foot of width" is, however, somewhat unrealistic and involves very low traffic figures. The 10-foot width, which approximates the width of a lane as usually defined, has therefore been adopted as the most convenient unit of measure. It should be borne in mind, however, that the approach width used by traffic entering an intersection from one direction on a two-way street is normally one-half of the total street width unless there are safety zones or islands.

To summarize briefly the foregoing, **the capacity of signalized intersections is expressed as vehicles per 10 feet of width per hour of green.**

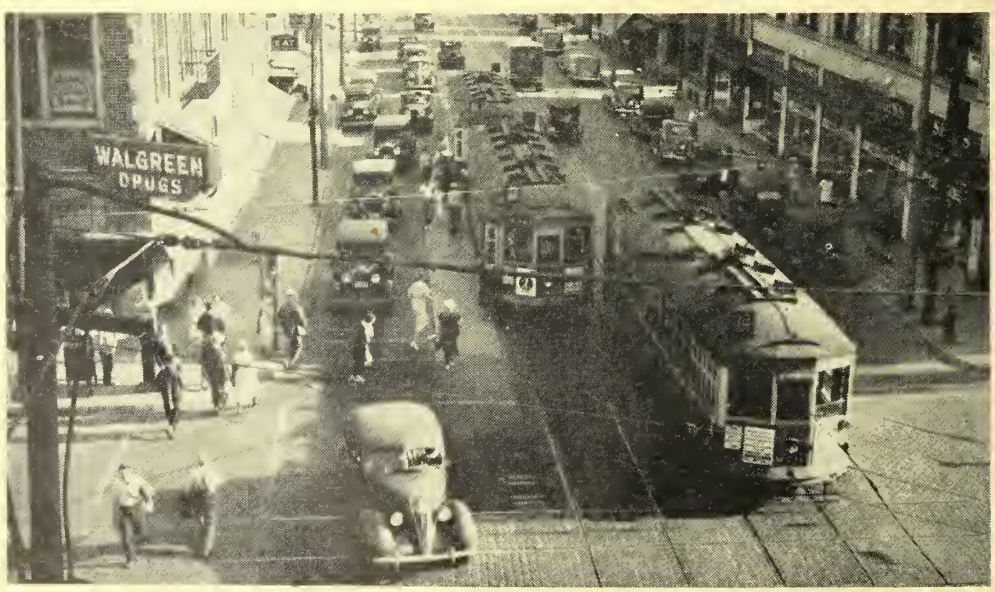
CAPACITY CLASSIFICATION

The problems encountered in moving traffic through intersections at grade vary in complexity from those usually found in rural areas, where the only interference to traffic is created by the cross movement of vehicles, to those found in urban areas, where there are pedestrians, cars entering and leaving the traffic stream on the approaches, and busses topping to discharge or pick up passengers.

Basic Capacity

With a little imagination, it is possible to visualize a rural intersection with no interference to traffic except that created by the cross movement of vehicles. If this cross traffic is properly controlled by a traffic signal, the burden of the interference is transferred to that signal. The condition then existing is comparable to that at a signal installation in an isolated area with no cross movement, the only interference being that resulting from the periodic interruption to traffic when the signal turns red for traffic on the highway. This is the type of intersection where basic capacity can be realized and, as such, is worthy of brief consideration to discover how traffic performs at this ultimate rate of flow.

In part IV of this report it was stated that the basic capacity of a 12-foot traffic lane on a multilane highway is 2,000 passenger cars per hour. The inference might be that the basic capacity of a similar traffic lane at a



A streetcar loading island. Pedestrians crossing to the island are doing so in violation of the red signal indication.

signalized intersection is 2,000 vehicles per hour of green because under ideal conditions the signal would seem to be the only impediment, and that the detrimental effect of the red indication could be eliminated by converting the rate of flow to vehicles per hour of green time. This deduction is not entirely correct, however, because one of the conditions that must prevail for this high volume to be realized is the movement of all vehicles at a uniform speed of about 30 miles per hour. This uniform speed cannot be achieved if any single unit of the traffic stream is stopped for any reason.

With a low traffic volume, many vehicles will approach the intersection while the signal is green and proceed through at a reasonably high rate of speed. With heavier volumes of traffic, an increasingly large number of vehicles will be stopped by the red indication. Irrespective of the traffic volume, some vehicles will slow down when turning, or for pedestrians, thus affecting the whole traffic stream.

Parked vehicles deprive a street of a much greater portion of usable width than the space they actually occupy. This street is in an "intermediate area."

When the volume approaches the possible capacity of the intersection so that every green interval is fully utilized, there will always be some vehicles waiting to enter the intersection as the light turns green. Under this condition, a relatively high percentage of the cars approaching the intersection will be required to stop even on a street with a synchronized or progressive signal system.

With most vehicles coming to a stop, the average speed at which they can cross an intersection is 10 to 15 miles per hour. With traffic moving at a speed of 12 miles per hour, the highest rate at which a single line of vehicles can enter an intersection is 1,500 passenger cars per hour. As a confirmation of this figure, experience has shown that the minimum spacing between passenger cars as they start from a standing position one behind the other in a 12-foot lane averages about 2.4 seconds. The time intervals for the first two vehicles in line are usually considerably greater than 2.4 seconds, but between succeeding vehicles the interval decreases progressively until it reaches an average minimum of 2.1 seconds between the fifth and sixth cars in line.

Within the range of vehicles normally accommodated by one signal cycle, 2.4 seconds is a representative average headway. This headway corresponds to an hourly rate of 1,500 vehicles. **It follows, then, that the basic capacity of an isolated signalized intersection is about 1,500 passenger cars per hour of green per 12-foot lane, or 1,250 vehicles per 10 feet of width per hour of green.** This figure corresponds to the highest rates at which vehicles have been observed to pass a point on a roadway removed from the influence of intersections after being stopped. It also corresponds to the rates attained for each movement through a signalized intersection where there are separate signal indications and traffic lanes for each traffic movement, and few pedestrians, so that the permitted movements do not interfere with one another.

High traffic density on a one-way street in a downtown area. Where busses continuously load and unload, a negligible number of passenger cars utilize the first lane.

Possible Capacity

Basic capacity can be attained only under the ideal conditions previously described. Inherent in any traffic stream, however, are several causes for minor delays, such as stalled motors and inept drivers, which cannot be eliminated by any practical means.

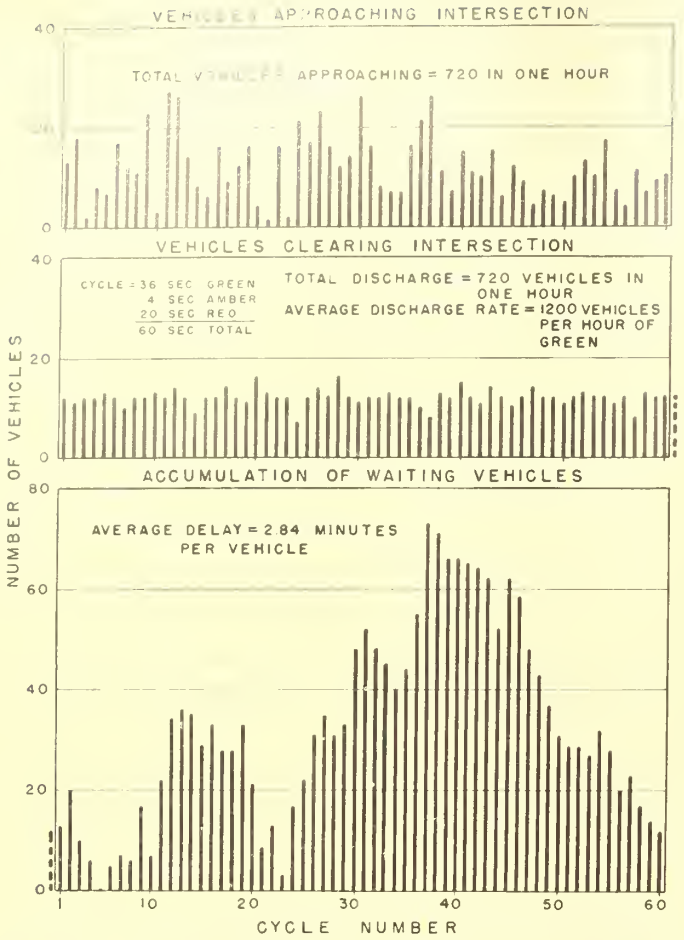

VEHICLES APPROACHING INTERSECTION
TOTAL VEHICLES APPROACHING = 720 IN ONE HOUR
VEHICLES CLEARING INTERSECTION
CYCLE = 36 SEC GREEN
4 SEC AMBER
20 SEC REQ.
60 SEC TOTAL
TOTAL DISCHARGE = 720 VEHICLES IN ONE HOUR
AVERAGE DISCHARGE RATE = 1200 VEHICLES PER HOUR OF GREEN
ACCUMULATION OF WAITING VEHICLES
AVERAGE DELAY = 2.84 MINUTES PER VEHICLE
NUMBER OF VEHICLES
CYCLE NUMBER

Figure 21.—Operation of traffic at a rural intersection loaded to its possible capacity.

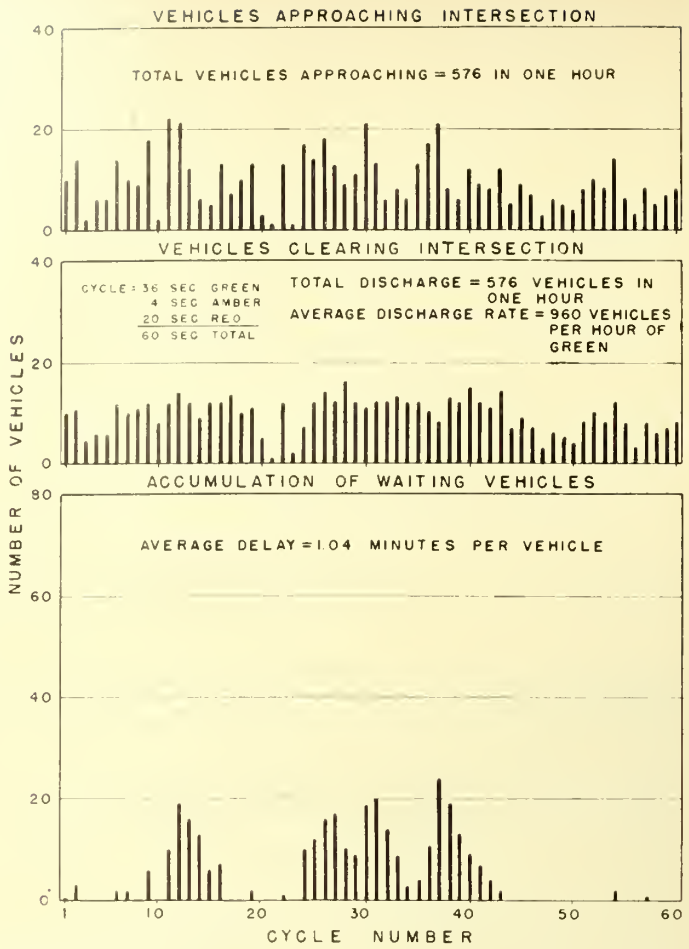

VEHICLES APPROACHING INTERSECTION
TOTAL VEHICLES APPROACHING = 576 IN ONE HOUR
VEHICLES CLEARING INTERSECTION
CYCLE = 36 SEC GREEN
4 SEC AMBER
20 SEC REQ.
60 SEC TOTAL
TOTAL DISCHARGE = 576 VEHICLES IN ONE HOUR
AVERAGE DISCHARGE RATE = 960 VEHICLES PER HOUR OF GREEN
ACCUMULATION OF WAITING VEHICLES
AVERAGE DELAY = 1.04 MINUTES PER VEHICLE
NUMBER OF VEHICLES
CYCLE NUMBER

Figure 22.—Operation of traffic at a rural intersection loaded to its practical capacity.

These conditions are included with the ideal in determining basic intersection capacity.

Basic capacity, however, does not include any reduction in capacity caused by the following, which are the principal factors that tend to reduce traffic flow at intersections:

1. Parked vehicles and vehicles being parked or leaving parking areas.
2. Turning movements.
3. Commercial vehicles, including streetcars.
4. Pedestrian interferences.
5. Inclement weather conditions.

When the basic capacity of an intersection is reduced by the effect which each of these factors that are present has on the traffic flow, the possible capacity of the intersection is obtained. **The possible capacity of an intersection approach, therefore, is the maximum number of vehicles that actually can be accommodated under the prevailing conditions with a continual backlog of waiting vehicles.**

Practical Capacity

When the traffic volume on any intersection approach is sufficient to tax the approach to its possible capacity during every signal interval for an hour—that is, when there is always a backlog of waiting vehicles—there will be times when the queue of waiting traffic will become extremely long, resulting in lengthy and intolerable delays for a large number of drivers.

This is illustrated graphically by figure 21, which shows the length of back-up, in terms of numbers of vehicles, that occurred at an intersection approach loaded to its possible capacity. Traffic approaching the intersection from one direction was of sufficient volume to utilize the full green period during each signal cycle for a full hour. Some drivers in this particular example were required to wait through six signal cycles before reaching and proceeding through the intersection. Such lengthy delays are intolerable and the traffic volume at which they occur can certainly not be considered practical. Yet, because of the normal short-time variation in traffic flow on approach highways or streets, this is the typical condition which must and does occur at an intersection when each green period of a fixed-time signal is fully utilized for an entire hour.

The practical capacity of an intersection approach is the maximum volume that can enter the intersection from that approach during 1 hour with most of the drivers being able to clear the intersection without waiting for more than one complete signal cycle. With the normal short-time variation in flow, practical intersection capacities have been found to be approximately 80 percent of the possible capacities. In other words, at most intersections few vehicles will be required to wait for more than one signal cycle if the hourly approach volume is 80 percent of the possible capacity. The comparatively little delay when the approach volume is 80 percent of the possible capacity is illustrated by figure 22 in which the approach volume is 80 percent of the approach volume shown in figure 21. It should be noted that all of the green periods were not fully utilized under the conditions represented by figure 22.

Many attempts to determine intersection capacities have been based on rates attained during green periods that were fully utilized. Other attempts have been made by expanding the average rate attained during the five or ten green periods in 1 hour when the largest number of vehicles passed through the intersection. The first method completely ignores the normal short-time fluctuation in traffic flow, whereas the second method fails to recognize the effect of inept drivers and a variety of other factors that are ever present in any traffic stream and cannot be eliminated. As a result, theoretical intersection capacities are calculated which are much higher than those possible of attainment even under the best of prevailing conditions.

The results obtained by either of these two methods are highly factitious and approach the basic capacity of intersections. Before these extremely high volumes could be attained in practice it would be necessary to eliminate those factors that cause any reduction in flow

below the rates attained during short peak periods. Among these factors are the stalling of motors, stopping to pick up passengers, and traveling at spacings greater than those used by the small group of reckless drivers.

CLASSIFICATION OF INTERSECTION TYPES

When the engineers throughout the country were asked to furnish data on observed intersection volumes that could be used to determine intersection capacities, they were requested to submit the highest traffic count made at each intersection during a period of 1 hour while most of the green intervals for at least one approach were fully utilized and a high percentage of the drivers had to wait for at least one full signal cycle before entering the intersection. It is evident, therefore, that the data upon which the following results are based represent volumes exceeding the practical capacities. It is not likely, however, that all the data represent possible capacities with all green intervals for an hour fully utilized. This condition rarely occurs even at the most congested intersections. The average condition represented is, therefore, somewhere between the possible and practical capacities. Were this not true for data obtained at the most congested intersections throughout the country, there would be no traffic problem at rural or urban intersections.

One hour is generally the shortest period of time that conforms to current traffic-counting practice. The hourly rates shown as a result of these investigations, therefore, include the short-time variations that normally occur in the flow of traffic. When applying the results to intersections where there are exceptionally heavy surges of short duration or where the flow actually exists for less than an hour, such as at an entrance or exit at a factory, the capacity rates must be calculated for a like fractional part of an hour.

A street intersection in a downtown area during a peak period.

This intersection, in an intermediate area, is loaded beyond its practical capacity. Parking is not permitted on the approach in this block.

It is extremely difficult to isolate each of the variable factors that reduce traffic flow at signalized intersections and to determine the extent to which each influences the capacity of intersection approaches of different widths. Efforts to segregate their effect one from the other are often frustrated by the fact that they are in many cases very closely interrelated as, for example, the number of right turns and the number of pedestrians crossing the intersecting street. Even where the factors affecting capacity can be segregated and evaluated, the different conditions under which they may later be applied are so varied in character that the resulting estimates of capacity should be considered as approximations only.

For example, in the discussion of turning movements which appears later, each 1 percent of left-turning vehicles is said to reduce the capacity of an intersection approach by 1 percent. Although this factor is correct for the average intersection under average condi-

An intersection in an outlying area, with added turning lanes for left-turn movements and separated turning lanes for right-turn movements.

An intersection in a downtown area of a small city during an off-peak period when parking was permitted.

tions, its application will only produce reasonably accurate results for all conditions.

Precise results cannot be expected in a broad application to intersections of varied width and having different types of traffic, for many reasons. Among these are:

1. If two or more successive vehicles desire to turn left, the effect per vehicle on the capacity of the street is not as great as if the vehicles turned at more widely spaced intervals. The larger the number of left-turning vehicles, the lesser the effect per vehicle.

2. The effect on capacity of left-turning vehicles is related to the number of oncoming vehicles going straight through and turning left.

3. Each left-turning vehicle crosses the path of pedestrians moving with the green light, and therefore the effect of a left turn is to some extent dependent on the number of pedestrians.

4. A vehicle waiting to make a left turn causes a greater relative reduction in capacity on a narrow street than on a wider street or on one having a wide center-dividing island.

It is obvious that the correction factors needed to meet the possible combinations of the five elements enumerated under the discussion of possible capacity would total a very sizable number. Rather than attempting to apply each of these varied correction factors, it is much more feasible that intersections be grouped into certain classes or categories for which the combined effect of several elements which usually occur in about the same combination can be easily determined or applied. The capacity of any intersection that does not fall directly within a particular class can be determined with a reasonable degree of accuracy by interpolation.

The variables by which intersections have been classified are as follows:

1. *Street width:* Various ranges of street width within which the capacity per 10 feet of width shows little variation when other conditions are comparable.

2. *Type of area:* Downtown, intermediate, or outlying.

3. *Parking regulations:* Parking prohibited or permitted.

4. *Streetcars:* With or without streetcars.

These variables are sufficiently descriptive to classify most intersections. It would have been desirable to have had further breakdowns for items such as the percentage of commercial vehicles, the presence of bus-loading zones, the percentage of vehicles involved in turning movements, the extent to which the parking spaces were utilized, and the number of pedestrians. The four variables listed above, however, are sufficiently descriptive to classify most intersections so that the actual capacity for any condition can be estimated with a reasonable degree of accuracy. Within the downtown area, for example, most available parking spaces will be utilized and there will generally be a large number of pedestrians during the peak hour. If this is not true at the particular intersection under consideration, a capacity either in the upper or lower part of the range for the downtown intersections would be more applicable than the average value.

An intersection on a high-type facility. The traffic volume is near the practical capacity of the intersection, although between intersections the highway is adequate for a much greater volume.

Table 17.—*Average of maximum observed capacities at urban signalized intersections free from streetcars and curb parking on the approach streets: For approaches loaded beyond their practical capacities*

	Downtown area	Intermediate area	Outlying area
VEHICLES PER HOUR OF GREEN PER 10 FEET OF APPROACH WIDTH [1]			
Total street width, curb to curb:			
31–46 feet	717	888	703
47–64 feet	740	595	638
65 feet and over	704	538	
All widths	730	695	673
AVERAGE PREVAILING CONDITIONS			
Pedestrian crossings	2,200	880	200
Percentage of vehicles:			
Turning right	12	11	11
Turning left	13	10	13
Percentage of commercial vehicles	11	16	9

[1] Approach width assumed to be one-half of total street width because these results include two-way streets only.

HIGHEST OBSERVED VOLUMES FOR AVERAGE CONDITIONS

Capacities where parking is prohibited

Table 17 shows the average traffic volumes recorded at a large number of intersections in cities distributed throughout the country, during periods in which the volumes approached the capacity of the intersections. These results are for four-way intersections controlled by fixed-time signals and free from streetcars and parked vehicles on the approach streets. The range of volumes averaged in table 17 is shown in figure 23.

The difference between the figures shown in table 17 and the basic intersection capacity of 1,250 vehicles per hour of green per 10 feet of width can be attributed to the combined effect of pedestrians, turning movements, commercial vehicles, short-time fluctuations in flow, and other causes as they were present at the average intersection of each type. In downtown areas the reduction below basic capacity from these causes averages about 40 percent, while in intermediate and outlying areas it is slightly larger, being about 43 percent.

In the downtown areas, there is little difference in the volume of traffic handled per unit of width by the narrow and the wider streets, whereas in the intermediate and outlying areas the narrower streets handled somewhat more traffic per unit of width than the wider streets. A number of explanations can be offered for this characteristic difference between street widths in the various areas, but such explanations are of little practical value unless the effect of each cause can be established and applied when estimating capacity volumes for specific intersections. The averages of the more important conditions affecting intersection capacities that prevailed at locations in each area where parking is prohibited are shown at the bottom of table 17.

Capacities where parking is permitted

Table 18 shows the average traffic volumes recorded at intersections where parking was permitted at the curb on the approach streets. Except for this feature, these intersections are comparable in all other respects to the intersections for which data are shown in table 17. It is emphasized that the rates are for the entire approach width, including the portion of the street occupied by the parked vehicles. This basis for showing the results was used so that the total effect of the parked vehicles could be obtained without introducing errors by deducting an assumed street width for the space they occupy. The range of observed volumes averaged in table 18 is shown in figure 23.

Table 18.—*Average of maximum observed capacities at urban signalized intersections free from streetcars but with curb parking permitted on the approach streets: For approaches loaded beyond their practical capacities*

	Downtown area	Intermediate area	Outlying area
VEHICLES PER HOUR OF GREEN PER 10 FEET OF APPROACH WIDTH [1]			
Total street width, curb to curb:			
31–46 feet	383	435	732
47–64 feet	410	475	618
65 feet and over	460	470	---
All widths	398	453	692
AVERAGE PREVAILING CONDITIONS			
Pedestrian crossings	2,700	500	200
Percentage of vehicles:			
Turning right	17	6	14
Turning left	9	8	11
Percentage of commercial vehicles	11	10	12

[1] Approach width assumed to be one-half of total street width because these results include two-way streets only

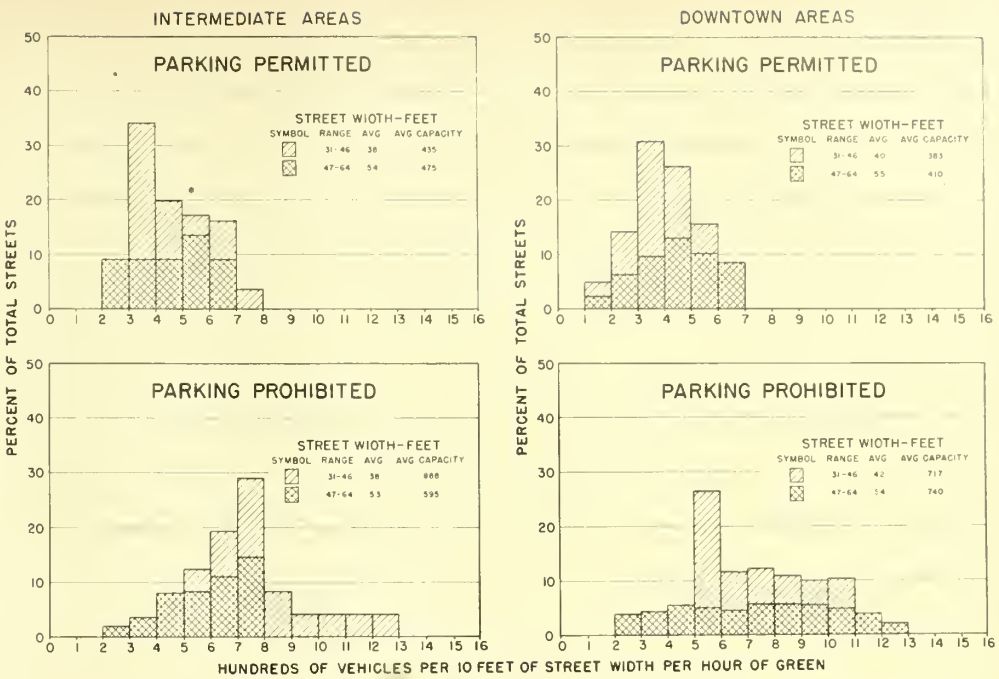


Figure 23.—Frequency distribution of intersection capacities.

Comparison of the data shown in tables 17 and 18 reveals that capacity volumes on downtown streets where parking is permitted are about 55 percent, on the average, of those observed where parking is prohibited. In intermediate areas the effect of curb parking is somewhat less, causing an average reduction of about 35 percent. In outlying areas, where but few vehicles park, street capacity is generally not impaired by vehicles parking or unparking. The occasional driver who does desire to park is usually able to pull directly into a parking space instead of backing in as is required in areas where most parking spaces are occupied.

Heavy bus traffic in a downtown area. At this intersection busses load and unload at both the curb and the outer separator.

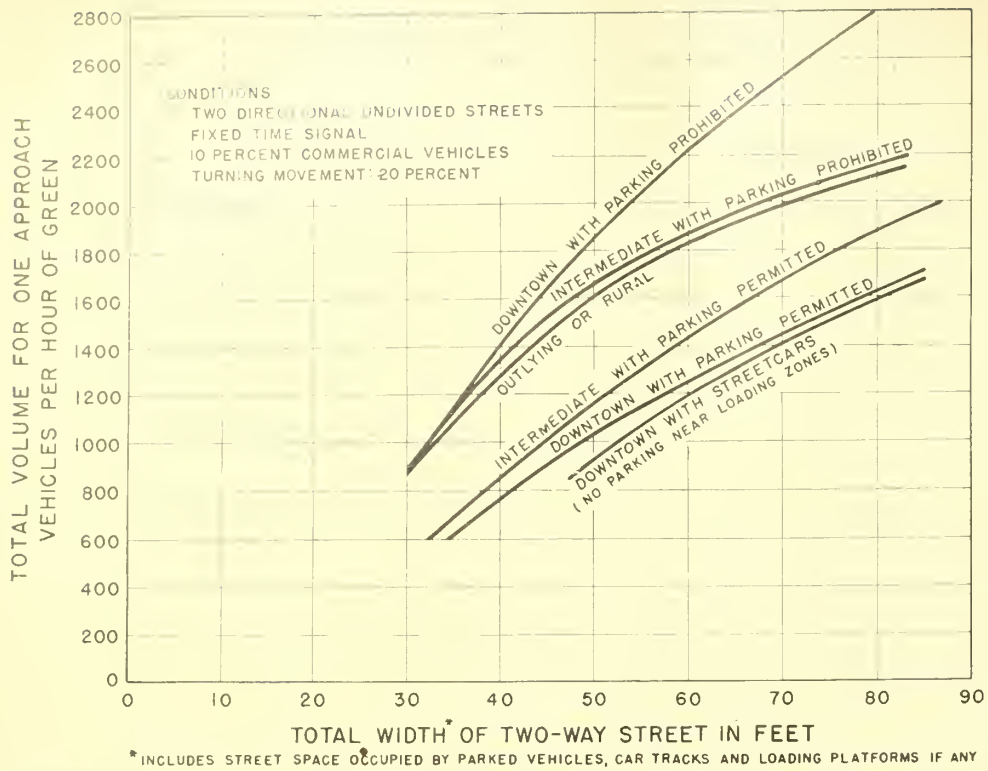

CONDITIONS
TWO DIRECTIONAL UNDIVIDED STREETS
FIXED TIME SIGNAL
10 PERCENT COMMERCIAL VEHICLES
TURNING MOVEMENT: 20 PERCENT
DOWNTOWN WITH PARKING PROHIBITED
INTERMEDIATE WITH PARKING PROHIBITED
OUTLYING OR RURAL
INTERMEDIATE WITH PARKING PERMITTED
DOWNTOWN WITH PARKING PERMITTED
DOWNTOWN WITH STREETCARS (NO PARKING NEAR LOADING ZONES)
TOTAL VOLUME FOR ONE APPROACH VEHICLES PER HOUR OF GREEN
TOTAL WIDTH* OF TWO-WAY STREET IN FEET
*INCLUDES STREET SPACE OCCUPIED BY PARKED VEHICLES, CAR TRACKS AND LOADING PLATFORMS IF ANY

Figure 24.—Average reported intersection capacities for two-way streets by type of area and parking regulation.

Curb parking has less effect on street capacities in intermediate areas away from business centers than in downtown areas because fewer cars are found at the curb even though it may be perfectly legal to park. Also the turnover is not as great, resulting in less interference to the moving traffic.

It is especially significant that whatever the width of a downtown street, intersection capacities where curb parking is permitted are from 43 to 47 percent below the capacities for similar streets where curb parking is prohibited. The difference for the two conditions is not due entirely to the street space occupied by the parked vehicles. Also included is the combined effect of the many other factors that tend to restrict traffic flow which are present where parking is permitted, such as the interference to through traffic caused by drivers maneuvering to enter or leave parking spaces and the tendency of drivers to shy away from parked vehicles to avoid the possibility of hitting a car starting to pull out, or a pedestrian coming from between the parked vehicles.

That a line of vehicles parked adjacent to the curb reduces the effective street width more than 7 or 8 feet becomes evident when street widths necessary to accommodate the same traffic volumes at locations where parking is permitted and where parking is prohibited are compared. Figure 24 shows, for example, that the average 40-foot street with no parking in a downtown area can accommodate the same volume of traffic as the average 68-foot street on which parking is permitted. This is a reduction of 14 feet in the effective width of the 68-foot street for each line of parked vehicles.

Figure 24 shows graphically the relation between the average reported intersection approach capacities and the total street width for the three types of areas. The conditions for which the curves apply are listed on the chart. It will be noted that these conditions are representative of those occurring at the average intersection, a separate curve being shown for streets on which streetcars operate.

The conditions listed on figure 24 are not all of those prevailing that affected the maximum observed volumes during periods that traffic flow exceeded the practical capacities. In the average downtown area, for example, traffic regulations are more rigidly enforced, available parking spaces are more fully utilized, and there is more stopping to unload and pick up passengers, than in the intermediate areas. The differences between these conditions in the two types of areas account in part at least for the higher average capacities on streets of the same widths in the intermediate areas than in the downtown areas when parking is permitted, whereas the reverse is true for the streets when parking is prohibited. In other words, street capacities are increased less by prohibiting parking in areas where a low demand for parking exists and parking restrictions are not enforced, than in areas where the demand is great and parking restrictions are enforced.

Table 19.—*Average reported capacities of downtown streets at intersections where there are two car tracks and a loading island with no parking adjacent to the island*

Street width		Vehicles per hour of green per 10 feet of width	
Curb to curb	Curb to island	Based on total width of approach	Based on width between curb and platform
Feet	*Feet*		
58	14	408	857
64	17	403	760
70	20	385	685
90	30	383	575

Capacity of streets having streetcars

The presence of streetcars can affect the capacity of a street in a number of ways, depending on local practices and regulations as well as on structural features such as loading islands. Headway between streetcars and the question of whether or not free-wheeled vehicles are permitted use of the car-track lanes are matters that must be considered in addition to all the elements that pertain to the capacity of streets where there are no streetcars.

The following characteristics are peculiar to most downtown intersections where congestion is a problem and where there are streetcar tracks:

1. Loading islands or safety zones are provided on the intersection approach.
2. Free-wheeled vehicles are not permitted to use the car-track lane adjacent to the loading island.
3. Curb parking is not permitted for a distance equal to and usually in excess of the length of the loading island.

Table 19 shows average reported capacities for downtown intersection approaches where the foregoing conditions prevail. These figures are for straight-through tracks only. If the intersection is blocked by streetcars turning, the time during which free-wheeled vehicles are prevented from moving for this reason should be deducted from the available green signal time. The range of volumes averaged in table 19 is shown in figure 25.

It will be noted that the capacity per 10 feet of width decreases rapidly with an increase in the distance between the curb and

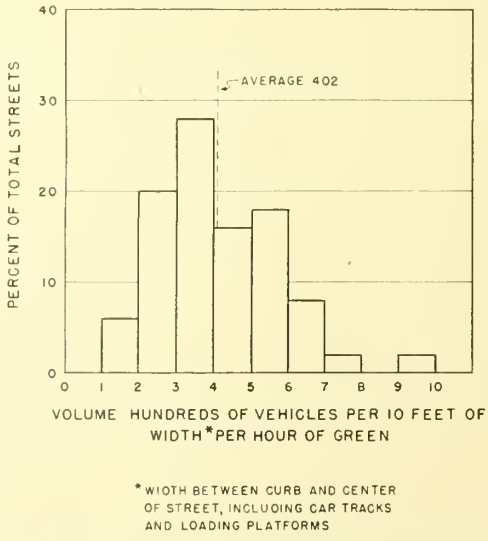

AVERAGE 402
PERCENT OF TOTAL STREETS
VOLUME HUNDREDS OF VEHICLES PER 10 FEET OF WIDTH* PER HOUR OF GREEN
*WIDTH BETWEEN CURB AND CENTER OF STREET, INCLUDING CAR TRACKS AND LOADING PLATFORMS

Figure 25.—Frequency distribution of observed intersection capacities on downtown streets with streetcars.

the loading platform. This suggests that pedestrians going to and from the platform may interrupt the flow of traffic to a greater extent when the street width between the curb and platform is wide than when it is narrow.

EFFECT OF VARIABLE CONDITIONS ON CAPACITY

Among the variable conditions that affect capacity of intersections are the proportion of commercial vehicles in the total traffic volume, the existence of bus stops, the extent of turning movements, the type of signal system, and the use of one-way operation. These conditions, and their effects on the capacity of intersections of the common type, are considered in the following paragraphs. Thereafter, the capacity of intersections on high-type facilities is discussed.

Common-Type Intersections

Commercial vehicles

The presence of commercial vehicles tends to reduce intersection capacities in terms of the total number of vehicles because their acceleration rates are lower and they occupy more road space than passenger cars. On the average, one commercial vehicle, not including those that stop to pick up or discharge passengers or goods, is equivalent in an intersection capacity sense to two passenger cars. This is true only when they are not involved in turning movements or where the turning movements can be made without more than normal interference with other vehicles.

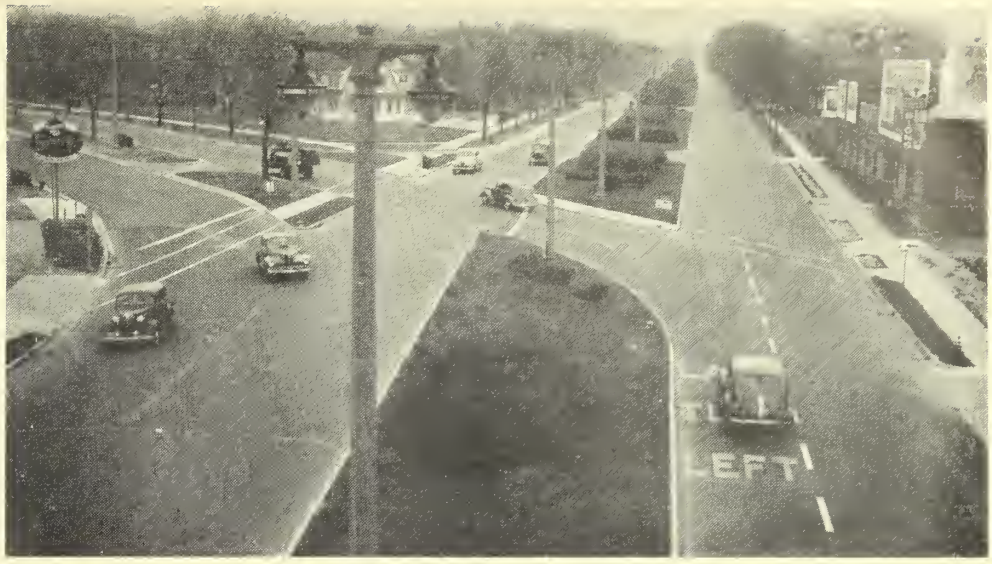
Bus stops

The effect of busses stopping to load and unload passengers on the capacity of a specific intersection depends to a large extent on the many different conditions present at the intersection. Analyses of the available data show the following results for the average conditions prevailing at signalized intersections where the total street width is under 60 feet:

1. Streets in the downtown and intermediate areas where parking is permitted, except at a near-side bus stop, have a 12-percent higher capacity than the streets where busses do not load and unload and parking is not restricted near the intersections.

2. In the downtown and intermediate areas where parking is prohibited, the streets with bus stops on the near side of the intersection have a 15-percent lower capacity than the streets where there are no bus stops. The corresponding figures for far-side bus stops are 8 percent in the downtown areas and 20 percent in the intermediate areas.

There is evidence, therefore, that elimination of parking near the intersection to provide a bus stop helps more to increase the capacity than the stopping of the busses decreases the capacity. Also, on streets where parking is prohibited, the results indicate that to achieve high capacities bus stops in the downtown area should be on the far side of the intersections wherever possible while stops outside of the central business district should be on the near side of the interseetions. It is recognized that considerations other than street capacity often govern in the selection of bus stop location. Insufficient data are available to draw any conclusions for streets over 60 feet wide or for the relative merits of near- and far-side stops on streets where parking is permitted.



A Y intersection on an expressway.

The above figures apply only to conditions found at the average intersection on streets in the two types of areas. There are many specific conditions that are far from average. Probably the one condition which is apt to be farthest from average at a specific location is the number of busses. A few studies conducted on the same streets during periods of normal operation and again during periods when the bus operators were on strike show that one bus has the same effect on intersection capacity as three to five passenger cars. These results apply only to locations where parking was prohibited for the entire distance between intersections and relatively short stops were made by the busses to pick up and discharge passengers.

Observations also indicate that on wide multilane streets where there are a large number of busses, or the busses stop for long periods to take on and discharge passengers, the busses in each direction reduce the effective street width for passenger cars by at least 12 feet.

Turning movements

The extent to which turning movements reduce the capacity of intersections is dependent upon such conditions as the intersection lay-out or treatment, pedestrian movement, the volume of oncoming traffic, and, of course, upon the number of vehicles turning right or left. In most instances, the detrimental effect of turning movements in rural areas can be greatly reduced through proper channelization and widening of the pavement. Under adverse conditions, the effect of turning movements may be sufficiently great to reduce the practical capacity of an intersection on a two-lane road or street by as much as 50 percent. On a multilane facility under similar conditions, the left lane may have its capacity reduced by 50 percent or more as a result of heavy turning movements.

The only general criterion obtained by analyzing the available intersection data was

An intersection with a separated right-turning lane.

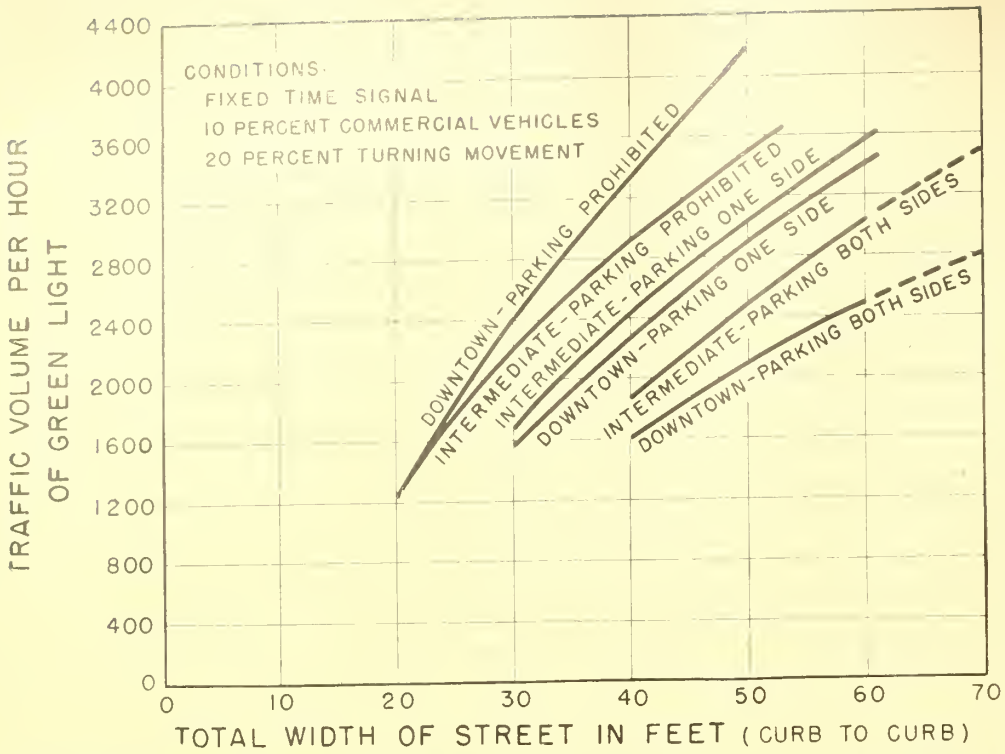


Figure 26.—Intersection capacities of one-way streets by type of area and parking regulation

that each 1 percent of the total traffic turning right reduced the capacity flow ½ percent, and that each 1 percent of the total traffic turning left reduced the capacity flow 1 percent. These percentages apply only where there is no separate signal indication for the turning movements and where all turns comprise less than 30 percent of the total traffic. Above 30 percent, no additional decrease in volume was apparent.

Where there are separate signal indications for turning movements, careful investigations are usually required and it is difficult to formulate specific rules for these situations. As a general rule, the total street width is proportioned between the various movements in the ratio of the number of lanes utilized by traffic in the performance of these movements. The capacity for each separate movement is then computed, using as a basis the length of the interval and width of street allotted to the particular movement. The capacity of the intersection approach is a summation of the computed capacities for all movements emanating from that approach. Because the object of separate signal indications is the elimination of conflicts between various movements, the rate of flow for turns will generally be the same as that for straight-through movements.

Signal system

The type of signal system on a street makes little difference in the possible capacity of the facility. Although a perfectly planned progressive signal system may function well with low or even moderately heavy traffic volumes, some traffic will be stopped by the red light at each signal when the traffic volume becomes sufficient to tax the capacities of the intersections. Vehicles following on the next band will be slowed by these waiting vehicles and the system is immediately thrown off balance. The higher speed for which the system was planned cannot be regained until the traffic density becomes less than that which occurs when the street is carrying near capacity loads. The benefits accruing to vehicle operators as a result of a progressive signal system are reflected in a saving in time to the traffic using the facility, but this time saving declines rapidly when the practical capacity is exceeded.

The saving in travel time with a progressive system is not directly related to the ability of the street to handle traffic in terms of the number of vehicles. Normally, the actual number of vehicles that can pass a point during 1 hour is not increased appreciably by a progressive system. The exception is where the distance between traffic signals is so short that the road space between them will not accommodate the number of vehicles passing through one intersection and headed for a second intersection during one complete signal cycle. This being true, there is a possibility that sounder principles may be developed for adjusting a progressive system for *peak loads* as a result of these investigations.

One-way operation

Figure 26 shows intersection capacities for one-way streets by type of area and parking regulation when 10 percent of the traffic is commercial vehicles and 20 percent of the total traffic is involved in turning movements. The advantage of one-way streets over two-way streets from the capacity viewpoint is illustrated for certain conditions by figures 27 and 28. The advantage of one-way streets over two-way streets will vary with the distribution of traffic by direction on the two-way streets, the relative number of turning movements involved, and the width of the streets.

Figure 27 shows the advantage of one-way streets over two-way streets in the downtown area where two-way streets normally carry between 50 and 60 percent of the traffic in one direction during the peak periods. Except for the streets over 60 feet wide with parking on both sides and a 50–50 distribution of traffic by direction on the two-way streets, the capacity of a one-way street is higher than the capacity of a two-way street. For example, a one-way street 20 feet wide with parking prohibited has approximately double the capacity of a two-way street with parking prohibited when between 50 and 60 percent of the traffic on the two-way street is traveling in the one direction. Likewise, a street 50 feet wide will accommodate between 45 and 60 percent more traffic with one-way operation than with two-way operation, depending on the distribution of traffic by direction on the two-way street.

Also, a one-way street with parking on one side will accommodate approximately the same total traffic as the same width of two-

The effect of left-turning vehicles on intersection capacities is dependent upon a large number of variable conditions.

way street with no parking. The two-way streets are considered to be at their capacity volumes when the traffic in the direction of the heavier flow equals the capacity of the approaches used by traffic traveling in that direction.

Figure 28 shows the comparative capacities of one-way and two-way streets in intermediate areas where the volume in one direction includes up to 67 percent of the total flow on two-way streets. For this condition there is also a greater advantage in making the narrow streets one-way than in making the wider streets one-way. It may be noted that the total capacity of a two-way street with a 67–33 distribution of traffic by directions is the same when parking is prohibited as when parking is permitted on the side carrying the lower traffic flow. Also, a one-way street 50 feet wide with parking on both sides has the same capacity as a 50-foot two-way street either with no parking or with parking on one side.

Intersections on High-Type Facilities

At highway intersections where pedestrian interference for the most part is eliminated, where separate lanes for each traffic movement are provided so that the permitted movements do not interfere with one another, where commercial vehicles are not present, and where the geometric features of the highway are of a high standard, a basic capacity of about 1,500 vehicles per hour of green per 12-foot lane may be approached. The type of facility to which this is applicable would be a divided highway, in a rural or an urban area, with added turning lanes of adequate design provided for both right and left turns, with shoulder space for disabled or temporarily stopped vehicles, and with no other roadside interference on the approaches of the intersection—that is, a facility normally referred to as an expressway.

Where such conditions exist, the method for capacity analysis given for street intersections is not directly applicable. Instead, a procedure of starting with the basic capacity and reducing this value to fit prevailing roadway and traffic conditions is suggested. Because express-type facilities are intended for rapid vehicular movement with a minimum of operational delay, practical capacity rather than possible capacity should be used for design. Under the conditions outlined in the preceding paragraph there is little difference between basic and possible capacity. Since practical capacity at intersections has been found to be about 80 percent of possible capacity, it follows that the practical capacity of a 12-foot traffic lane is $1{,}500 \times 0.80 = 1{,}200$ passenger vehicles per hour of green. This is equivalent to 1,000 passenger vehicles per hour of green per 10 feet of width.

ADJUSTMENTS FOR SPECIFIC CONDITIONS

The ability of intersections to accommodate high hourly traffic volumes has been grossly overrated in the past. The application of attainable capacities, which in some cases are so low as to be astonishing, will be an important step in alleviating congestion on existing facilities and in estimating the needed improvement for future requirements.

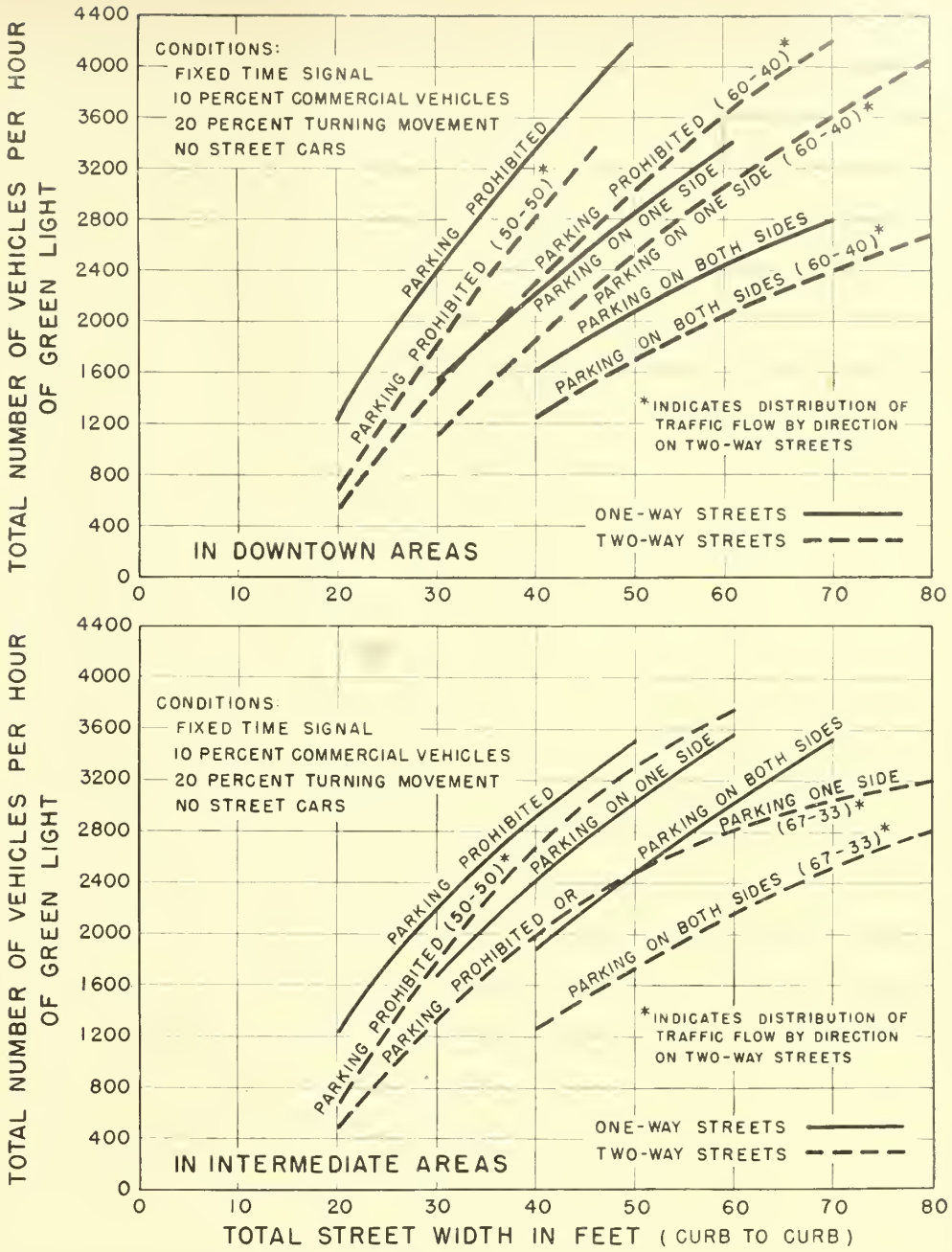
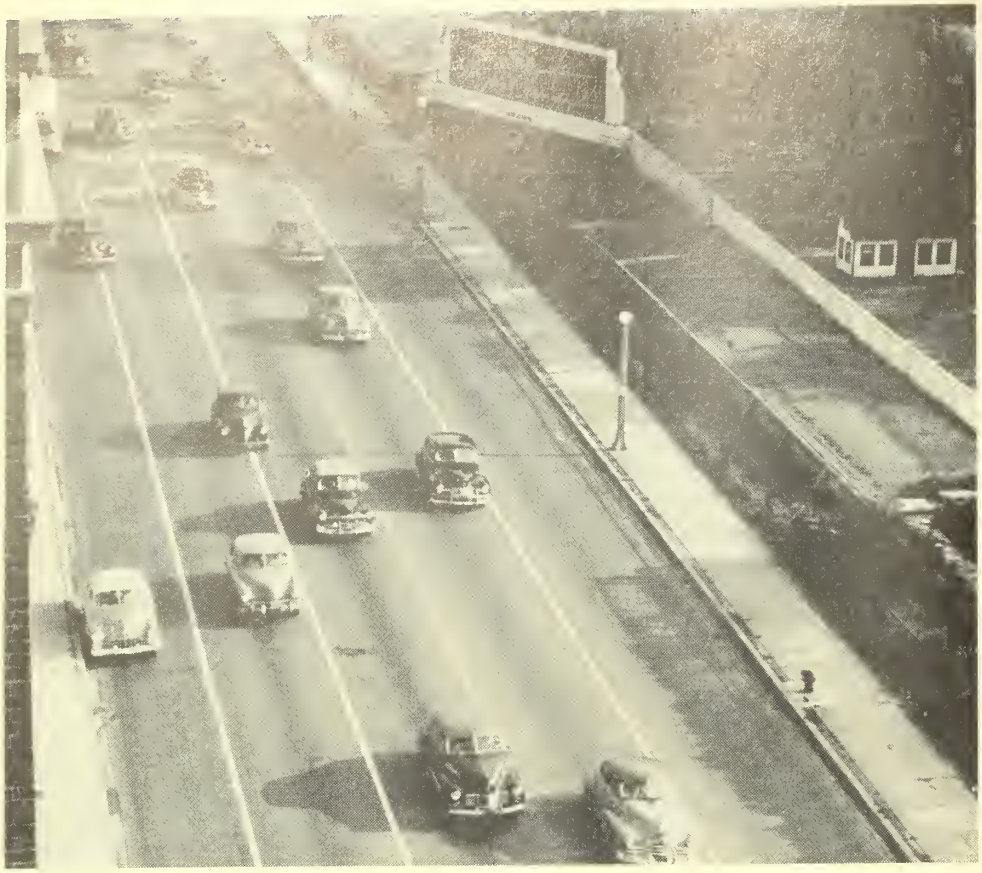


Figure 27 (above) *and figure 28* (below).—*Comparison of average street intersection capacities with one-way and two-way operation, in downtown and in intermediate areas.*

As previously stated, the intersection capacities that have been presented are averages for maximum recorded traffic volumes under the conditions that prevailed at the average intersection of each general class. They are peak volumes during periods that the approaching traffic exceeded the practical intersection capacities. Since there is a wide range in the maximum volumes for each group into which the intersections have been classified, it is essential, when estimating the capacity of any specific intersection, that the average figures as presented be adjusted to provide for the difference between average conditions and the existing or future conditions at the specific location. Figures 23 and 25, which show the distribution of observed rates at congested intersections, illustrate the need for making such adjustments.

Rates exceeding those for the average intersection are applicable only when conditions which restrict the flow of traffic are not present in the same degree as at the average intersection of the same class. Likewise, when the restricting conditions are more pronounced than at the average intersection, and this will be the case about 50 percent of the time, rates below average should be applied.

The highest rates recorded are about 700 vehicles per 10 feet of street width per hour of green or co time in areas where parking is permitted between intersections, and about 1,250 vehicles per 10 feet of street width per hour of green in areas where parking is prohibited. These rates cannot occur where the various movements interfere with one another, where pedestrians are present, or where



Flexible lane usage: Normally this is a two-way street, but during the morning rush there are four lanes provided for south-bound traffic and one lane, restricted to local and bus traffic, for north-bound vehicles. The operation is reversed for the afternoon peak load. Widely spaced vehicles suggest moderately high speeds, which is indicative of heavy volume with little congestion. Note that the single parked vehicle has deprived traffic of the use of a full lane and, further, that traffic passing the standing vehicle allows a wide margin of clearance.

trucks or busses are present. Even under the most ideal conditions for high capacities where these rates did occur, operating conditions could not be considered satisfactory.

Intersections on existing or contemplated facilities should not be expected to accommodate the peak capacities that have been recorded at the few locations where conditions were most favorable for high rates of flow. Such an expectation would probably result in operating conditions that would be less favorable than those now present in our most congested areas.

If it were possible to list all the factors that tend to reduce intersection capacities, and the quantitative effect of each, the most appropriate procedure to follow in estimating the capacity of a specific intersection would be to start with the capacity for ideal conditions and deduct a certain amount for each of the prevailing conditions that are not ideal. This procedure is not possible of application, however, because a quantitative measure has been obtained for only the more important factors that affect intersection capacity.

A number of adjustments are necessary when applying the information for average intersection conditions, as shown by figure 24, to a specific location where conditions are not average. The most important of these adjustments are described here.

I.—Two-way streets with no added turning lanes and no separate signal period for turning movements

1. Possible and practical capacities:

A. *Possible capacity.*—On an average, possible capacities are about 10 percent higher than the average rates represented in figure 24. Volumes 10 percent higher will pass through the intersection, but only with a continuous back-log of vehicles and extremely long delays to a high percentage of the drivers.

B. *Practical capacity.*—On an average, practical capacities are about 10 percent lower than the average rates represented in figure 24. Volumes 10 percent lower will pass through the intersection with few drivers having to wait longer than for the first green period.

2. Commercial vehicles:

Subtract 1 percent for each 1 percent by which commercial vehicles exceed 10 percent of the total number of vehicles, or add 1 percent for each 1 percent that commercial vehicles are less than 10 percent of the total.

3. Turning movements:

A. *Right turns.*—Subtract ½ percent for each 1 percent by which traffic turning right exceeds 10 percent of the total traffic, or add ½ percent for each 1 percent that traffic turning right is less than 10 percent of the total (maximum reduction for right turns not to exceed 10 percent).

B. *Left turns.*—Subtract 1 percent for each 1 percent by which traffic turning left exceeds 10 percent of the total traffic, or add 1 percent for each 1 percent that traffic turning left is less than 10 percent of the total (maximum deduction for left turns not to exceed 20 percent).

Note.—Maximum deduction for right and left turns combined should not exceed 20 percent.

4. Bus stops and elimination of parking near intersection:

A. *On streets where parking is prohibited.*—

(a) No bus stop: Add 5 percent.

(b) Bus stop on near side: Subtract 10 percent.

(c) Bus stop on far side: Subtract 3 percent in downtown areas and 15 percent in intermediate areas.

(d) Where the number of busses is so great that at least one is always loading or unloading, subtract 12 feet from each approach width for either near- or far-side stops when applying the curves in figure 24; then add the number of busses and make the adjustments for items 1, 2, and 3, above, but do not include the busses as commercial vehicles in item 2.

B. *On streets with bus stops and where parking is permitted except at the bus stop.*—

(a) With bus stop on near side: Add ¼ percent for each 1 percent of right and left turns, but maximum increase not to exceed 6 percent.

(b) With bus stop on far side: Make no correction.

(c) Where the number of busses is so great that at least one is always loading or unloading: Subtract 6 feet from the approach width for either near- or far-side stops when applying the curves in figure 24; then add the number of busses and make the adjustments for items 1, 2, and 3, above, but do not include the busses as commercial vehicles.

C. *On streets where parking is permitted and there are no bus stops.*[11]—Deduct ¼ percent for each 1 percent that right and left turns combined are of the total traffic, but maximum deduction not to exceed 6 percent. Then, if parking is prohibited more than 20 feet in advance of the cross walk, add

$$P\left(\frac{D-20}{5G}\right) \text{ percent, where}$$

P=total percentage of right and left turns, but not to exceed 30.

D=distance in feet that parking is prohibited in advance of cross walk, but not to exceed $5G$ plus 20.

G=seconds of green indication per signal cycle.

[11] The discussions concerning the needed length of no-parking zones in advance of cross walks, the capacity of added turning lanes (item II), the effect of separate signal indications (item III), and the procedure for determining capacities of intersections on one-way streets (item IV) and on high-type facilities (item V), represent a rationalization based on such facts and data as are available concerning these important subjects, for which the reported material is insufficient for statistical analysis.

D. *On streets where parking is eliminated for a limited distance on both sides of the intersection.*—Where parking is eliminated in advance of the intersection for a distance in feet equal to or greater than $5G$ and (1) parking is also eliminated beyond the intersection for a distance equal to or greater than $5G$, or (2) the street beyond the intersection widens at least one lane: Use the upper curve (parking prohibited) in figure 24.

II.—Two-way streets with added turning lanes but no separate signal indication for the turning movements [12]

1. Use the width of the through lanes as one-half of the street width when applying the curves in figure 24.

2. Then add 5 percent for an added right-turn lane, 10 percent for an added left-turn lane, or 15 percent when both right- and left-turn lanes are added to the normal width.

3. Then:

(a) *For a right-turn lane.*—Add the number of vehicles turning right but not to exceed either $600\times\frac{G}{C}$ vehicles per hour or $\frac{D-20}{25}\times N$ vehicles per hour, where

G=green interval in seconds.
C=total signal cycle in seconds.
D=length of added turning lane in feet.
N=number of signal cycles per hour.

(b) *For a left-turn lane.*—Add the number of vehicles turning left but not to exceed the capacity of the left-turn lane. The capacity of the left-turn lane per hour of green in terms of passenger cars may be estimated as the difference between 1,200 vehicles and the total opposing traffic volume per hour of green in terms of passenger cars, but not less than two vehicles per signal cycle.

4. Then adjust as in I–1 and I–2.

III.—Two-way streets with turning lanes and separate signal indication [12]

Streets with turning lanes and separate signal indication (also applies where left-turn lanes for opposite directions of travel are within the normal street width and both straddle the centerline); pedestrians controlled:

A. *With right-turn lane.*—

(1) Use the width of the through lanes as one-half of the street width when applying figure 24 and increase the rate of flow by 5 percent.

(2) Then add the number of vehicles turning right, but not more than 800 vehicles per 10 feet of width of turning lane per hour of separate green indication; adjust for possible or practical capacity as in item I–1 and for commercial vehicles as in item I–2.

B. *With left-turn lane.*—

(1) Use the width of the through lanes as one-half of the street width when applying figure 24 and increase the rate of flow by 10 percent.

(2) Then add the number of vehicles turning left, but not more than 800 vehicles per 10 feet of width of turning lane per hour of separate green indication; adjust for possible or practical capacity as in item I–1 and for commercial vehicles as in item I–2.

Note: Where conditions are such that through vehicles also use turning lanes, it is more appropriate to apply figure 24 to (1) entire approach width for period of left-turn indication and (2) width of through lanes for balance of through period. Add results and make normal adjustments except that for left turns 10 percent is always added.

IV.—One-way streets [13]

Figure 26 shows hourly intersection capacities for urban one-way streets by type of area and parking regulation under average conditions. The most important adjustments for conditions that are not average are:

1. Possible and practical capacities:
Same as for item I–1.

2. Commercial vehicles:
Same as for item I–2.

3. Turning movements:
Subtract ½ percent for each 1 percent by which the combined traffic turning right and left exceeds 20 percent of the total traffic, or add ½ percent for each 1 percent that it is below 20 percent (maximum deduction for turns not to exceed 20 percent).

4. Bus stops and elimination of parking near the intersection:
Same as for item I–4.

5. Added turning lanes:
Use the normal street width when applying the curves in figure 26. Before applying items 1 and 2 above:

(a) Add 5 percent for an added right- or left-turn lane or 10 percent if both a right- and left-turn lane have been added.

(b) For a right- (or left-) turn lane add the number of vehicles turning right (or left) but not to exceed either $600\times\frac{G}{C}$ vehicles per hour or $\frac{D-20}{25}\times N$ vehicles per hour.

V.—High-type facilities [13]

Where conditions exist similar to those previously described for intersections on high-type facilities, the following procedure for estimating practical or design capacities should be used:

1. Through movement:
Use 1,000 vehicles per hour of green per 10 feet of lane width. Deduct 1 percent for each 1 percent that commercial vehicles are of the total through movement during the peak hour.

2. Turning movements on added lanes:
To determine the total capacity of an intersection approach, add the number of vehicles turning right and left during the hour to the capacity of the through flow; however, each turning volume should not exceed the capacity of a turning lane as determined below. Also, the added turning lane should be of sufficient length to accommodate at least twice the average number of turning vehicles that would accumulate during the red interval.

A. *Right turns on same signal indication with through movement.*—

(1) Where there is no adjacent frontage road and no pedestrian interference, use capacity of turning lane as 1,000 vehicles per hour of green per 10 feet of width. Deduct 1 percent for each 1 percent that commercial vehicles are of the right-turning traffic during the peak hour.

(2) Where right turns are in conflict with frontage-road traffic, use capacity of right-turning lane as the difference between 1,200 vehicles and the total conflicting traffic volume (expressed in terms of passenger vehicles) on the adjacent frontage road per hour of green; adjust this difference by deducting 1 percent for each 1 percent that commercial vehicles are of the right-turning traffic during the peak hour. The volume thus determined must not be greater than that estimated under item (1) above, but not less than two vehicles per signal cycle.

(3) Where right turns are in conflict with pedestrian movements on the cross street, reduce flow estimated under item (1) above as follows:

Downtown area	20 percent.
Intermediate area	10 percent.
Outlying area	no reduction.

(4) Where right turns are in conflict with both frontage-road traffic and pedestrians, use the lower of the two values estimated under items (2) and (3) above.

B. *Left turns on same signal indication with through movement.*—Use capacity of left-turning lane as the difference between 1,200 vehicles and the volume of the opposing through traffic movement (expressed in terms of passenger vehicles) per hour of green; adjust this difference by deducting 1 percent for each 1 percent that commercial vehicles are of the left-turning traffic during the peak hour; minimum capacity not less than two vehicles per signal cycle.

C. *Added turning lanes on separate signal indication (pedestrians controlled).*—For either right or left turns, use capacity of 1,000 vehicles per hour of separate green indication per 10 feet of lane width; deduct 1 percent for each 1 percent that commercial vehicles are of the particular turning traffic during the peak hour.

3. Bus stops on added turning lane:

A. *Far-side bus stop.*—No apparent effect on capacity of the intersection approach.

B. *Near-side bus stop (no separate signal indication for right turns).*—Bus stops on the right-turning lane tend to reduce the capacity of the through movement by requiring some right-turning movements to be made around the bus directly from through traffic lanes. Capacity in such case may be adjusted as follows:

(1) Where the number of busses stopping during the peak hour is so great as to nullify the use of the added lane for right turns, deduct 1½ percent from the

[12] See footnote 11.

[13] See footnote 11.

(2) Where approximately one bus stops in the [illegible] per cycle, deduct 5 percent from the through flow for each 1 percent that the right-turning traffic is of the total traffic; then add the number of busses and right- and left-turning vehicles to obtain total practical capacity.

(3) Where busses stop less frequently than one about every fourth or fifth cycle, the effect of busses may be neglected.

APPLICATION OF INTERSECTION CAPACITY INFORMATION

When adjusting the volumes shown in figure 24 for conditions that are not average, each adjustment must be made as a separate step, using the result of the previous step for each consecutive adjustment. To accomplish this simply, when a number of adjustments are necessary, each adjustment can be calculated and added to or subtracted from 1.00, and a total factor then obtained from these individual factors by multiplying them together. The examples that follow illustrate correct applications of the data.

Example 1

Problem

What are the possible and practical capacities of one approach to an intersection on a two-way street, 45 feet wide from curb to curb, in a downtown area where parking is prohibited, 20 percent of the traffic turns right, 15 percent turns left, 5 percent of the total traffic is commercial during peak hours, there are bus stops on the near side of the intersection, and the traffic light has a GO or green period of 35 seconds out of the 60-second cycle?

Solution

From figure 24, the reported capacity of one approach on the average street 45 feet wide from curb to curb in a downtown area with parking prohibited is 1,660 vehicles per hour of green.

The following adjustments are required because conditions are not average:

Cause	*Effect*	*Factor*
Right turns	(10−20)÷= − 5%	0.95
Left turns	(10−15) = − 5%	0.95
Commercial vehicles	(10− 5) =+ 5%	1.05
Near-side bus stop	−10%	0.90
Total factor=0.95×0.95×1.05×0.90=		0.85

$$\text{Possible capacity} = 1.10 \times 0.85 \times \frac{35}{60} \times 1{,}660 = 905 \text{ vehicles per hour in the direction of the heavier flow.}$$

$$\text{Practical capacity} = 0.90 \times 0.85 \times \frac{35}{60} \times 1{,}660 = 711 \text{ vehicles per hour in the direction of the heavier flow.}$$

Example 2

Problem

What is the practical capacity of two intersecting streets, if the north-south street is 40 feet wide and the east-west street is 54 feet wide? Parking is permitted on the east-west street and banned on the north-south street. The intersection is in a downtown area where average conditions prevail and the signal cycle is 27 seconds green and 3 seconds amber on each leg, with a total of 60 seconds.

Solution

From figure 24, the practical capacity of the north-south street is $0.90 \times 1{,}410 = 1{,}269$ vehicles per hour of green. The traffic signal reduces this volume to $1{,}269 \times \frac{27}{60} = 571$ vehicles per hour in one direction.

From figure 24, the practical capacity of the east-west street is $0.90 \times 1{,}110 = 999$ vehicles per hour of green. This is reduced by the traffic signal to $999 \times \frac{27}{60} = 450$ vehicles per hour in one direction.

Example 3

Problem

What should be the distribution of signal time at the intersection described in example 2 if the peak hourly volume in one direction on the north-south street is 600 vehicles per hour and that on the east-west street is 400 vehicles per hour?

Solution

The minimum time in minutes per hour for each operation, assuming a 60-second cycle, is as follows:

	Minutes
Green, north-south street	$\frac{600}{1{,}269} \times 60 = 28.4$
Green, east-west street	$\frac{400}{999} \times 60 = 24.0$
Amber: 6 seconds per cycle	= 6.0
Total	58.4

The green period for the north-south street should be:

$$\frac{28.4}{(28.4+24.0)} \times (60-6.0) = 29.3 \text{ seconds.}$$

The green period for the east-west street should be:

$$\frac{24.0}{(28.4+24.0)} \times (60-6.0) = 24.7 \text{ seconds.}$$

Example 4

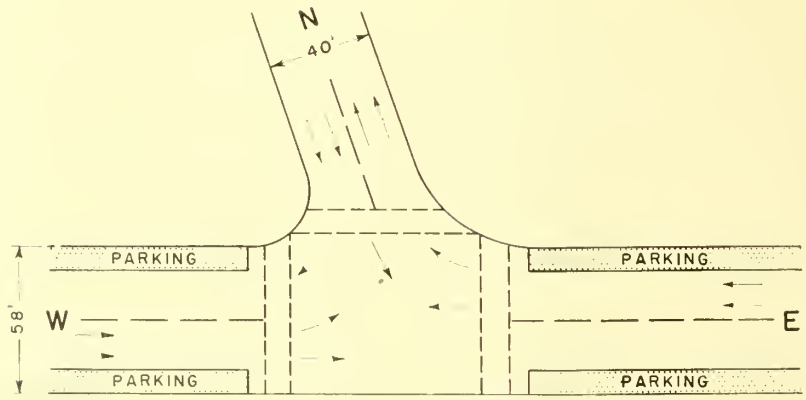
Problem

An intersection as described in example 3 is severely congested by volumes exceeding those used in that example. To relieve this situation it is proposed that parking be banned on both streets, and that the north-south street be widened. After these improvements, traffic volumes of 750 vehicles per hour in one direction on the east-west street and 900 vehicles per hour in one direction on the north-south street are anticipated. What should be the width of the north-south street and the signal timing for the intersection, using an 80-second cycle with 3 seconds of amber?

Solution

Total amber time per hour will be $6 \times \frac{3{,}600}{80} = 270$ seconds = 4.5 minutes.

A 54-foot street with no parking has a capacity of 2,000 vehicles per hour of green,



INTERMEDIATE AREA
FIXED TIME SIGNAL
2 PHASE CONTROL
CYCLE-70 SEC.

E-W STREET
GREEN INTERVAL - 40 SECONDS
COMMERCIAL VEHICLES - 12%
TURNING MOVEMENTS
W TO N - 11% OF APPR. VOL.
E TO N - 24% OF APPR VOL.
NO BUS STOPS

N APPROACH
GREEN INTERVAL - 25 SECONDS
COMMERCIAL VEHICLES - 15%
TURNING MOVEMENTS
N TO E - 65% OF APPR. VOL.
N TO W - 35% OF APPR. VOL.
NO BUS STOPS

Figure 29.—Illustrative example 5.

from figure 24. Thus the east-west traffic will require $\frac{750}{0.9\times2{,}000}\times60=25.0$ minutes. There remain 30.5 minutes or $\frac{30.5}{60}$ hours of green on the north-south street, permitting a practical capacity of $\frac{60}{30.5}\times900=1{,}770$ vehicles per hour of green. This practical capacity corresponds to a reported capacity for average conditions of $\frac{1{,}770}{0.9}=1{,}967$ vehicles per hour of green. A street 52 feet wide will accommodate this volume, as shown in figure 24.

The signal timing should be:

	Seconds
Green, north-south street $\frac{30.5}{(30.5+25.0)}\times(80-6.0)=40.7$	
Green, east-west street $\frac{25.0}{(30.5+25.0)}\times(80-60)=33.3$	
Amber $2\times3.0=$	6.0
Total	80.0

Example 5

Problem

What are the possible and practical capacities of each approach of the T intersection shown in figure 29? If the peak-hour traffic (on the basis of two-way flow) is 9 percent of the average daily traffic, and 60 percent of the peak-hour traffic is in one direction, what will be the average daily traffic on the east-west street in the block west of this intersection if it operates at possible capacity?

Solution

West Approach

From figure 24, the reported capacity of one approach on the average 58-foot street in an intermediate area with parking is 1,380 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−12) =−2%	0.98
Right turns	(10−0)½=+5%	1.05
Left turns	(10−11) =−1%	0.99
Parking not restricted at intersection (See adjustment I-4-C)	−(0+11)¼=−3%	0.97
Total factor=0.98×1.05×0.99×0.97=		0.99

Possible capacity $=1.10\times0.99\times\frac{40}{70}\times1{,}380=860$ vehicles per hour.

Practical capacity $=0.90\times0.99\times\frac{40}{70}\times1{,}380=700$ vehicles per hour.

East Approach

For average conditions, reported capacity is 1,380 vehicles per hour of green, as shown above.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−12) =−2%	0.98
Right turns	(10−24)½=−7%	0.93
Left turns	(10−0) =+10%	1.10
Parking not restricted at intersection	−6%	0.94
Total factor=0.98×0.93×1.10×0.94=		0.94

Possible capacity $=1.10\times0.94\times\frac{40}{70}\times1{,}380=810$ vehicles per hour.

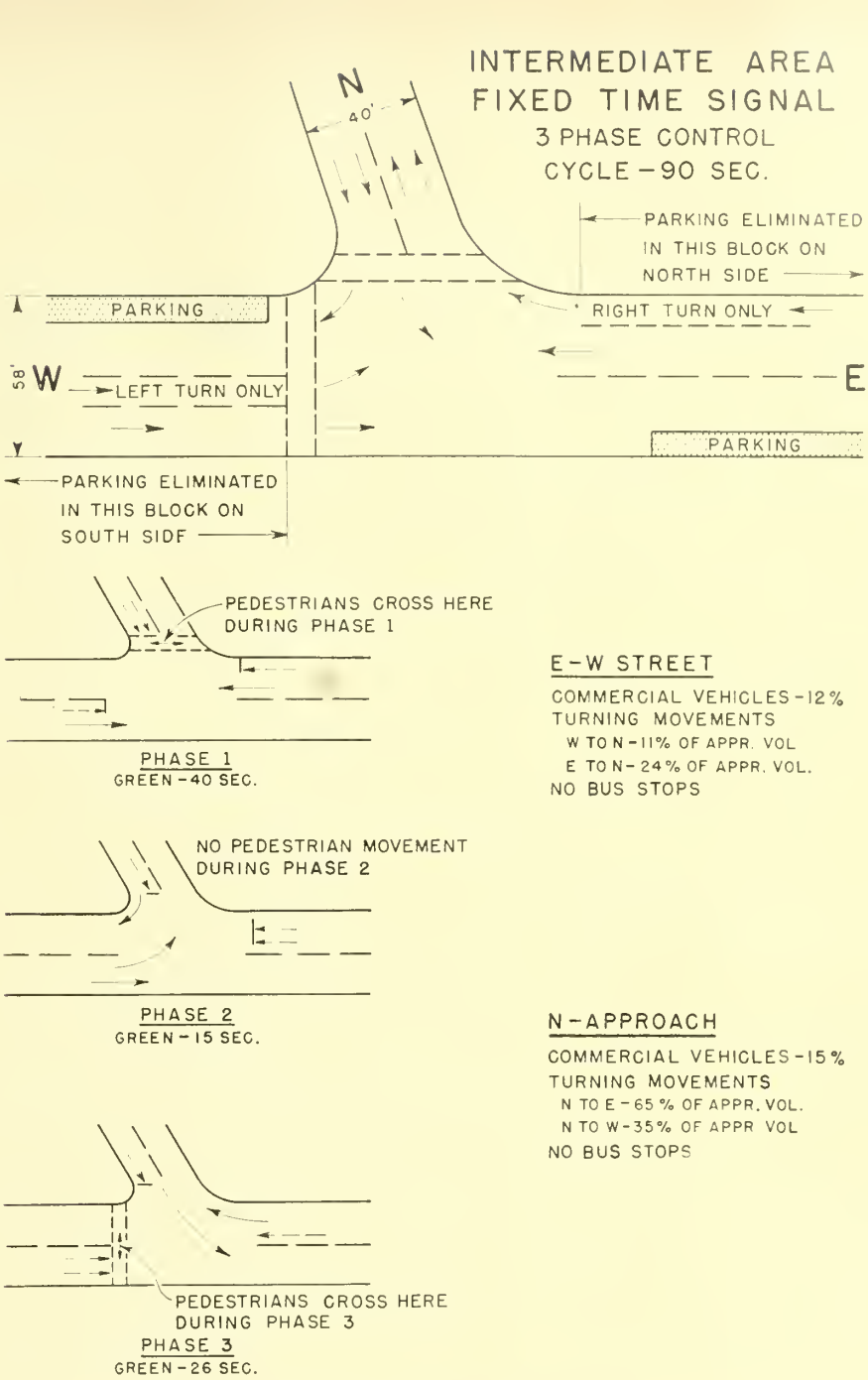


Figure 30.—Illustrative example 6.

Practical capacity $=0.90\times0.94\times\frac{40}{70}\times1{,}380=665$ vehicles per hour.

North Approach

On the intercepted street of a T intersection, the larger of the two turning movements is considered as the through movement; therefore, as indicated on figure 29, the through movement is 65 percent and the turning movement is 35 percent of the total flow during the peak hour.

From figure 24, the reported capacity for a 40-foot street under average conditions is 1,350 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−15) =− 5%	0.95
Right turns (maximum deduction)	−10%	0.90
Left turns (none)	(10− 0) =+10%	1.10
No parking and no bus stops	+ 5%	1.05
Total factor=0.95×0.90×1.10×1.05=		0.99

Possible capacity $=1.10\times0.99\times\frac{25}{70}\times1{,}350=525$ vehicles per hour.

Practical capacity $=0.90\times0.99\times\frac{25}{70}\times1{,}350=430$ vehicles per hour.

The average daily traffic on the east-west street in the block west of the intersection, when operating at possible capacity, is:

$$\left[860+\left(\frac{40}{60}\times860\right)\right]\times\frac{100}{9}=15{,}920 \text{ vehicles per day.}$$

Example 6

Problem

What are the possible and practical capacities of each approach of the intersection shown in figure 30? This is the same intersection as in figure 29 except for partial elimination of parking and substitution of multiphase

signal control [illegible] capacity of the east-west [illegible] and [illegible] vehicle-pedestrian conflicts.

Solution

West Approach

Through movement, phases 1 and 2.—Width available for through movement (parking eliminated as shown) $=\frac{58}{2}+10=19$ feet (see adjustment III B 1). Capacity for average conditions, from figure 24, for a street twice this width and with no parking is 1,300 vehicles per hour of green. The green interval for phases 1 and 2 = 40+3 (amber for opposing through movement) + 15 = 58 seconds.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−12) = − 2%	0.98
Right turns	(10−0)½ = + 5%	1.05
Left turns	(10− 0) = +10%	1.10
No bus stop	+ 5%	1.05
Total factor = 0.98×1.05×1.10×1.05 =		1.19

Possible capacity $=1.10\times1.19\times\frac{58}{90}\times1{,}300=$ 1,100 vehicles per hour.

Practical capacity $=0.90\times1.19\times\frac{58}{90}\times1{,}300$ = 900 vehicles per hour.

Left turning movement, phase 2.—Left turning is 11 percent of the total volume on the west approach, or $1{,}100\times\frac{11}{89}=135$ vehicles per hour when the approach is operating at possible capacity, and $900\times\frac{11}{89}=110$ vehicles per hour when operating at practical capacity.

Capacity of this 10-foot left-turn lane for 15-second separate green indication and with 12 percent commercial vehicles (factor 0.98):

Possible capacity $=1.10\times0.98\times800\times\frac{15}{90}=$ 145 vehicles per hour (see adjustment III-B-2).

Practical capacity $=0.90\times0.98\times800\times\frac{15}{90}=$ 120 vehicles per hour.

Volumes of 135 and 110, above, therefore do not exceed the capacity of the left-turning lane.

Total capacity of west approach—

Possible capacity = 1,100 + 135 = 1,235 vehicles per hour.

Practical capacity = 900 + 110 = 1,010 vehicles per hour.

East Approach

Through movement, phase 1.—Same as for west approach, except for length of green interval, which is 40 seconds:

Possible capacity $=1{,}100\times\frac{40}{58}=760$ vehicles per hour.

Practical capacity $=900\times\frac{40}{58}=620$ vehicles per hour.

Right-turning movement, phase 3.—Right turning is 24 percent of the total volume on the east approach $=760\times\frac{24}{76}=240$ vehicles per hour when the approach is operating at possible capacity, and $620\times\frac{24}{76}=200$ vehicles per hour when operating at practical capacity.

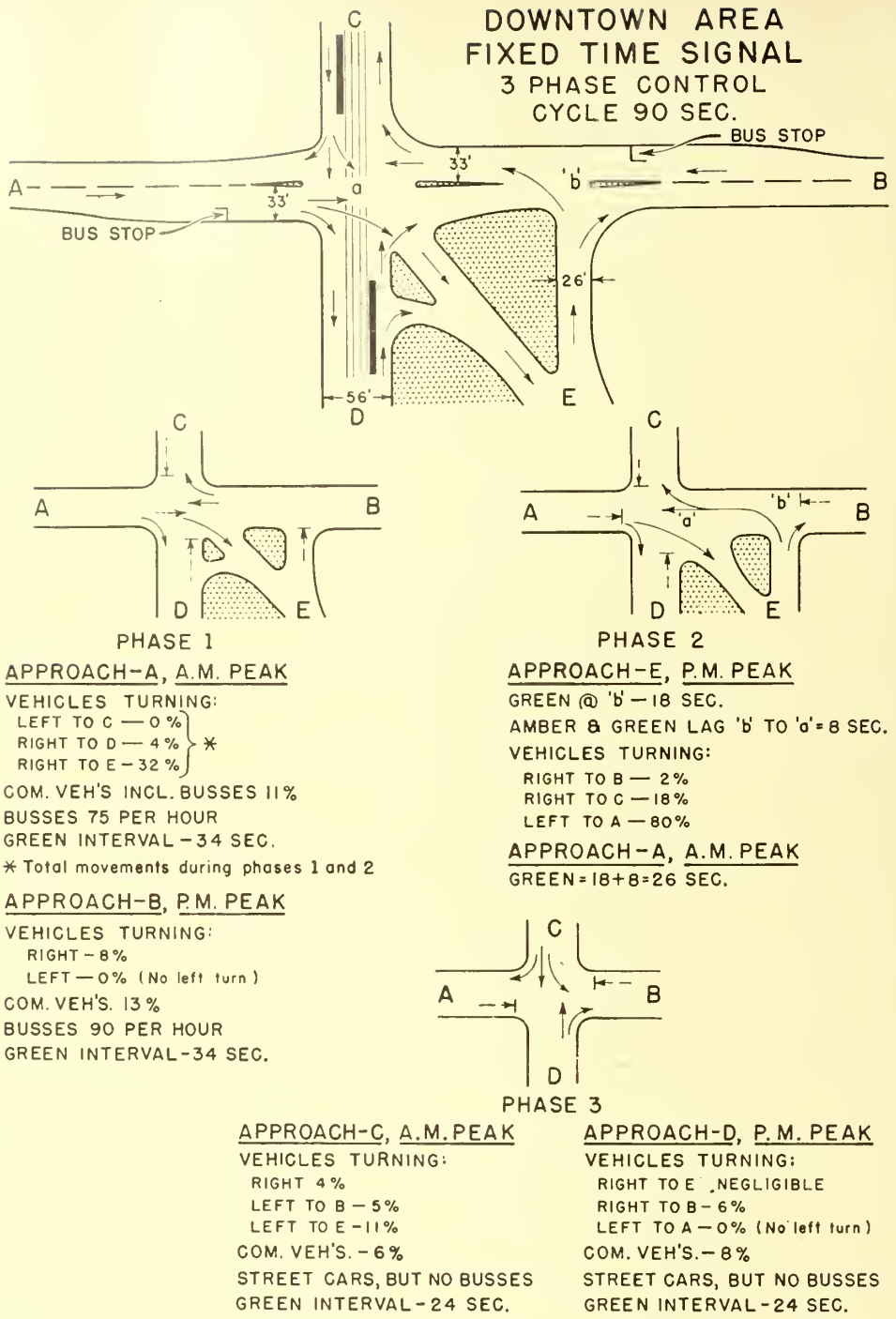


Figure 31.—Illustrative example 7.

Capacity of the 10-foot right-turn lane for 26-second separate green indication and with 12 percent commercial vehicles:

Possible capacity $=1.10\times0.98\times800\times\frac{26}{90}=$ 250 vehicles per hour.

Practical capacity $=0.90\times0.98\times800\times\frac{26}{90}=$ 210 vehicles per hour.

Volumes of 240 and 200, above, therefore do not exceed the capacity of the right-turn lane.

Total capacity of east approach.—

Possible capacity = 760 + 240 = 1,000 vehicles per hour.

Practical capacity = 620 + 200 = 820 vehicles per hour.

North Approach

Left-turning movement, phase 3.—Capacity of the 10-foot lane with 15 percent trucks and 26-second green interval:

Possible capacity $=1.10\times0.95\times800\times\frac{26}{90}=$ 240 vehicles per hour.

Practical capacity $=0.90\times0.95\times800\times\frac{26}{90}=$ 200 vehicles per hour.

Right-turning movement, phase 2.—Right turning is 35 percent of the total volume on the north approach, or $240\times\frac{35}{65}=130$ vehicles per hour when the approach is operat-

ing at possible capacity, or $200\times\frac{35}{65}=110$ vehicles per hour when operating at practical capacity.

Capacity of this 10-foot right-turn lane for 15-second separate green indication and with 15 percent commercial vehicles:

Possible capacity $=1.10\times0.95\times800\times\frac{15}{90}=$ 140 vehicles per hour.

Practical capacity $=0.90\times0.95\times800\times\frac{15}{90}=$ 115 vehicles per hour.

Volumes of 130 and 110, above, therefore do not exceed the capacity of the right-turn lane.

Total capacity of north approach.—

Possible capacity $=240+130=370$ vehicles per hour.

Practical capacity $=200+110=310$ vehicles per hour.

Example 7

Problem

What is the possible capacity of each approach of the multiple intersection shown in figure 31?

Solution

Approach *A*

Phase 1.—The effective width of approach $=33-12=21$ feet (12-foot deduction for at least one bus always loading or unloading during peak hour; see adjustment I–4–A–(d).

Capacity of approach for average conditions, from figure 24, for a street $2\times21=42$ feet wide is 1,510 vehicles per hour of green.

Total green interval per cycle for movements *A* to *D* and *A* to *E* is $34+26=60$ seconds (phase 1 plus phase 2).

During phase 1:

A to $D=\frac{34}{60}\times4\%=0.57\times4\%=2.3\%$ of total approach volume.

A to $E=\frac{34}{60}\times32\%=0.57\times32\%=18.2\%$ of total approach volume.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles [14]	(10−4) = + 6%	1.06
Right turns [15]	(10−2.3)½= + 4%	1.04
Left turns	(10−0) = +10%	1.10
Total factor = 1.06×1.04×1.10=		1.21

Possible capacity, phase $1=\left(1.10\times1.21\times\frac{34}{90}\times1{,}510\right)+75$ busses $=835$ vehicles per hour.

Phase 2.—During phase 2:

A to $D=\frac{26}{60}\times4\%=0.43\times4\%=1.7\%$ of total approach volume.

A to $E=\frac{26}{60}\times32\%=0.43\times32\%=13.8\%$ of total approach volume.

Total capacity, C, of approach *A* during phase 1 and phase 2 is composed of the following:

835 vehicles per hour = total, all movement, phase 1.

1.7% C = *A* to *D* movement, phase 2.

13.8% C = *A* to *E* movement, phase 2.

Then: $C=835+0.017C+0.138C$. Solving, $C=990$ vehicles per hour.

During phase 2:

A to $D=0.017\times990=$ 17 vehicles per hour.

A to $E=0.138\times990=136$ vehicles per hour.

Total ..... 153 vehicles per hour.

The total, and the movement from *A* to *E*, are less than the capacity of a turning lane (see adjustment III–A–(2).

Approach *E*

Because the street is actually two-way, with a channelizing island, a capacity value from figure 24 is applicable, rather than a value from figure 26, which is for one-way streets.

At point *b* consider left-turn movement to *A* as a through movement. Capacity of approach for average conditions from figure 24 for a street of $2\times26=52$ feet is 1,930 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−10) = 0%	1.00
(Since commercial vehicles were not reported, use average conditions)		
Right turns	(10− 2)½=+ 4%	1.04
Left turns	(10− 0) =+10%	1.10
No bus stop	+ 5%	1.05
Total factor = 1.00×1.04×1.10×1.05=		1.20

Possible capacity $=1.10\times1.20\times\frac{18}{90}\times1{,}930=$ 510 vehicles per hour.

At point *a* computations reveal a higher capacity. Point *b* therefore governs.

Approach *B*

Phase 1.—The effective width and capacity for average conditions are the same as for approach *A*.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles [16]	(10−2) =+ 8%	1.08
Right turns	(10−8)½=+ 1%	1.01
Left turns	(10−0) =+10%	1.10
Total factor = 1.08×1.01×1.10=		1.20

Possible capacity, phase $1=(1.10\times1.20\times\frac{34}{90}\times1{,}510)+90$ busses $=845$ vehicles per hour.

Phase 2.—From approach *E* (above), possible capacity $=510-(0.02\times510)=500$ vehicles per hour.[17]

Phase 1 plus phase 2.—The possible capacity of approach *B* $=845+500=1{,}345$ vehicles per hour.

Approach *C*

The capacity of the approach under average conditions for a street width of 56 feet with streetcars, from figure 24, is 1,100 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−6) =+4%	1.04
Right turns	(10−4)½ =+3%	1.03
Left turns	10−(10+6)=−6%	0.94
Total factor = 1.04×1.03×0.94=		1.01

[14] Within the limits of accuracy of the solution it may be assumed that busses (75 in number) comprise about 7 percent of the total approach volume. Since commercial vehicles, including busses, comprise 11 percent of the approach volume it follows that 4 percent of this traffic is made up of trucks, for which adjustment must be made. No adjustment is made for busses as commercial vehicles where the 12-foot deduction in street width is made: see adjustment I–4–A–(d).

[15] Movement *A* to *E* is considered as part of the through movement.

[16] Busses are assumed to be 11 percent of the total traffic approaching from *B*. Other commercial traffic is therefore 2 percent of the total: see footnote 14.

[17] The deduction made is for right turn *E* to *B*.

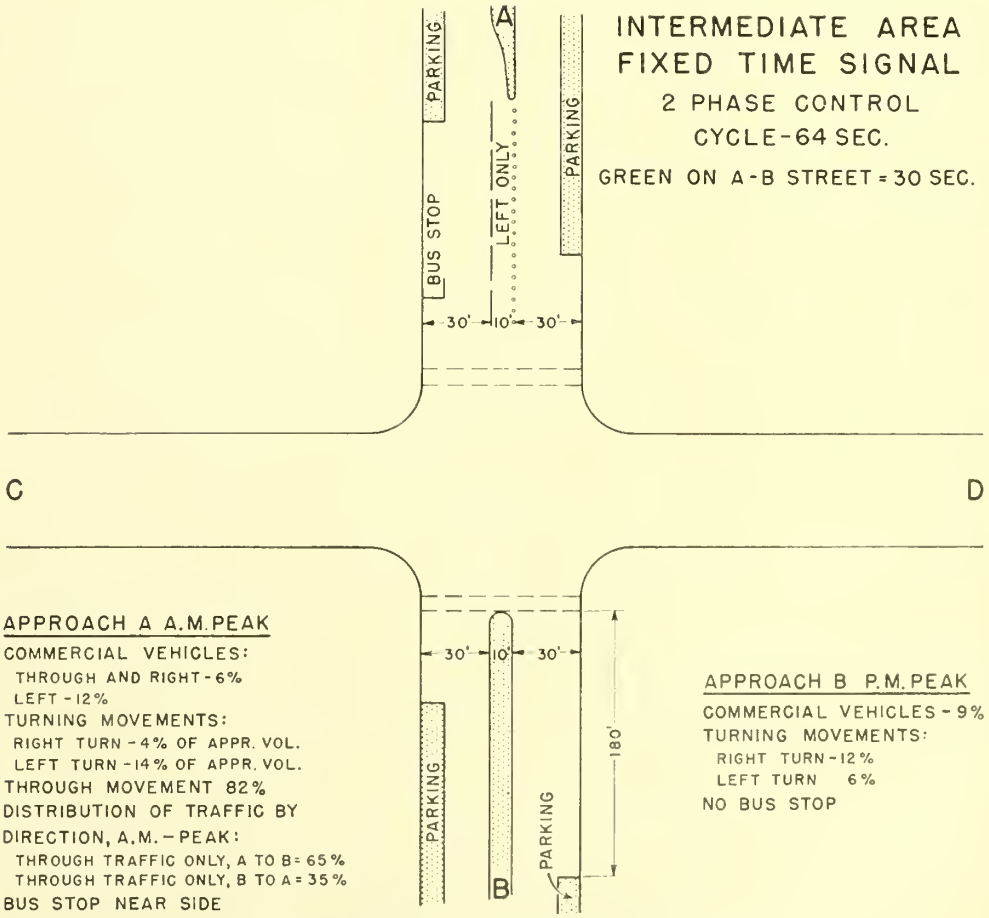


Figure 32.—Illustrative example 8.

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Possible capacity = $1.10 \times 1.01 \times \frac{24}{90} \times 1,100 =$ 325 vehicles per hour.

APPROACH *D*

As for approach *C*, the capacity for average conditions is 1,100 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−8) =+2%	1.02
Right turns	(10−6)½=+2%	1.02
Left turns	(10−0) =+10%	1.10
Total factor=1.02×1.02×1.10=		1.15

Possible capacity = $1.10 \times 1.15 \times \frac{24}{90} \times 1,100 =$ 370 vehicles per hour.

Example 8

Problem

What are the practical capacities of approaches *A* and *B* of the intersection shown in figure 32?

Solution

APPROACH *A*

Through and right-turn movements.—The reported capacity, from figure 24, for one approach on a street 30×2=60 feet wide with parking is 1,430 vehicles per hour of green (the left-turn lane is considered separately below).

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−6) =+ 4%	1.04
Right turns	(10−4)½=+ 3%	1.03
Left turns	(10−0) =+10%	1.10
(Left turns on added turning lane: See adjustment II-2)		
Bus stop	(4+6)¼=+ 1%	1.01
(See adjustment I-4-B-(a): Left turns are considered as zero in this computation because they are made from an added turning lane.)		
Total factor=1.04×1.03×1.10×1.01=		1.19

Practical capacity = $0.90 \times 1.19 \times \frac{30}{64} \times 1,430 =$ 720 vehicles per hour.

Left-turn movement only.—Left turns are 14 percent of the total approach volume, or $720 \times \frac{14}{86} = 117$ vehicles per hour; or $117 \times \frac{64}{30} =$ 250 vehicles per hour of green. With 12 percent commercial vehicles, this is equal to 280 equivalent passenger cars per hour of green.

Check for capacity of left-turn lane.—The opposing through movement, *B* to *A*, during the morning peak is 35 percent of the combined through movement in both directions:

Through movement *A* to *B* = $720 \times \frac{82}{(100-14)} = 685$ vehicles per hour.

Through movement *B* to *A* = $685 \times \frac{35}{65} = 370$ vehicles per hour, or $370 \times \frac{64}{30} = 790$ vehicles per hour of green.

With 9 percent commercial vehicles, this is equal to 860 equivalent passenger cars per hour of green.

The capacity of the left-turn lane (see adjustment II-3-(b) is 1,200 − 860 = 340 vehicles per hour of green. The left-turn movement of 280 passenger cars per hour of green, or 117 vehicles per hour, can therefore be accommodated.

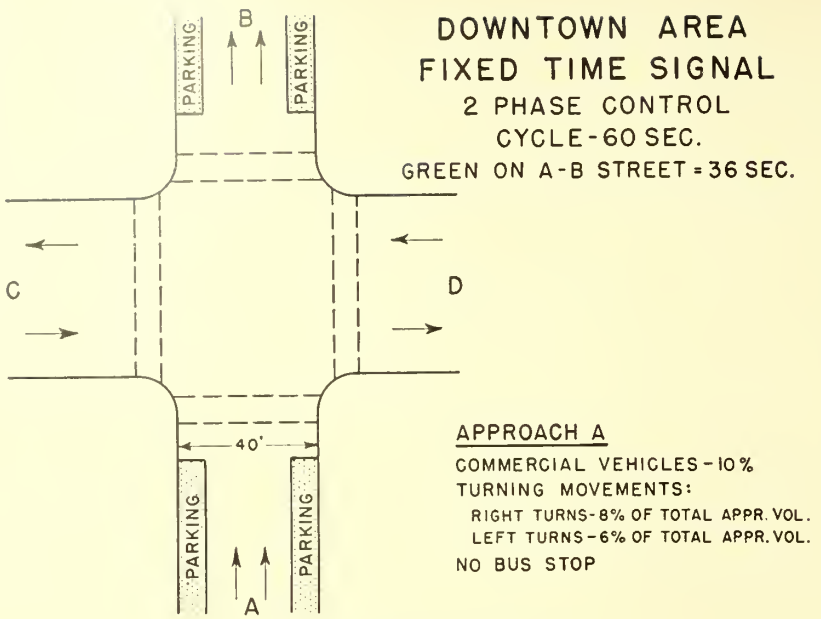


Figure 33.—Illustrative example 9.

Total practical capacity of approach A.—720+117=837 vehicles per hour.

APPROACH *B*

The capacity of approach *B*, from figure 24, is the same as that for approach *A*, or 1,430 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−9)= +1%	1.01
Right turns	(10−12)½= −1%	.99
Left turns	(10−6)= +4%	1.04
Parking restriction	(12+6)−¼(12+6)= +14%	1.14
(See adjustment I-4-C)		
Total factor=1.01×0.99×1.04×1.14=		1.19

Practical capacity = $0.90 \times 1.19 \times \frac{30}{64} \times 1,430$ = 715 vehicles per hour.

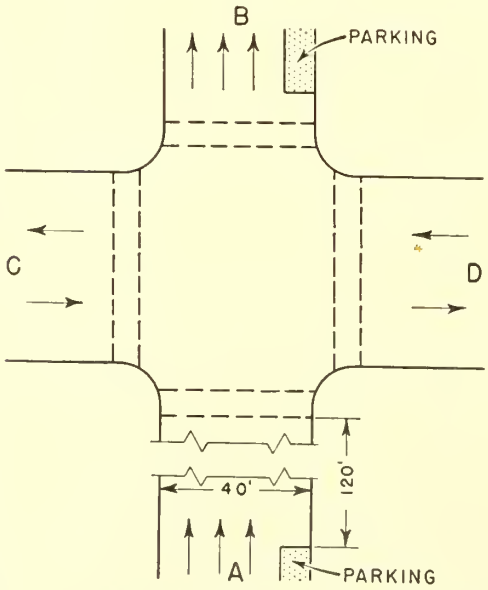


Figure 34.—Illustrative example 10: information as in figure 33, except for parking on right side only with 120-foot restriction.

Example 9

Problem

What is the practical capacity of the one-way street in figure 33?

Solution

From figure 26, the capacity when conditions are average is 1,620 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	(10−10) =0%	1.00
Right and left turns	(20−14)½=+3%	1.03
Parking not restricted at intersection	−¼×14 =−4%	0.96
(See adjustment I-4-C)		
Total factor=1.00×1.03×0.96=		0.99

Practical capacity = $0.90 \times 0.99 \times \frac{36}{60} \times 1,620$ = 870 vehicles per hour.

Example 10

Problem

What is the practical capacity of the street described in example 9, if parking is eliminated on the left and restricted 120 feet in advance of the cross walk on the right, as shown in figure 34?

Solution

From figure 26, the capacity when conditions are average is 2,250 vehicles per hour of green.

Adjustments:

Cause	*Effect*	*Factor*
Right and left turns	(20−14)½ =+3%	1.03
120-foot parking restriction	$(-\frac{1}{4} \times 14) + 14\left(\frac{120-20}{5 \times 36}\right)$	
(See adjustment I-4-C)	=+4%	1.04
Total factor=1.03×1.04=		1.07

Practical capacity = $0.90 \times 1.07 \times \frac{36}{60} \times 2,250$ = 1,300 vehicles per hour.

Example 11

Problem

What is the practical capacity of each intersection approach on the expressway shown in figure 35?

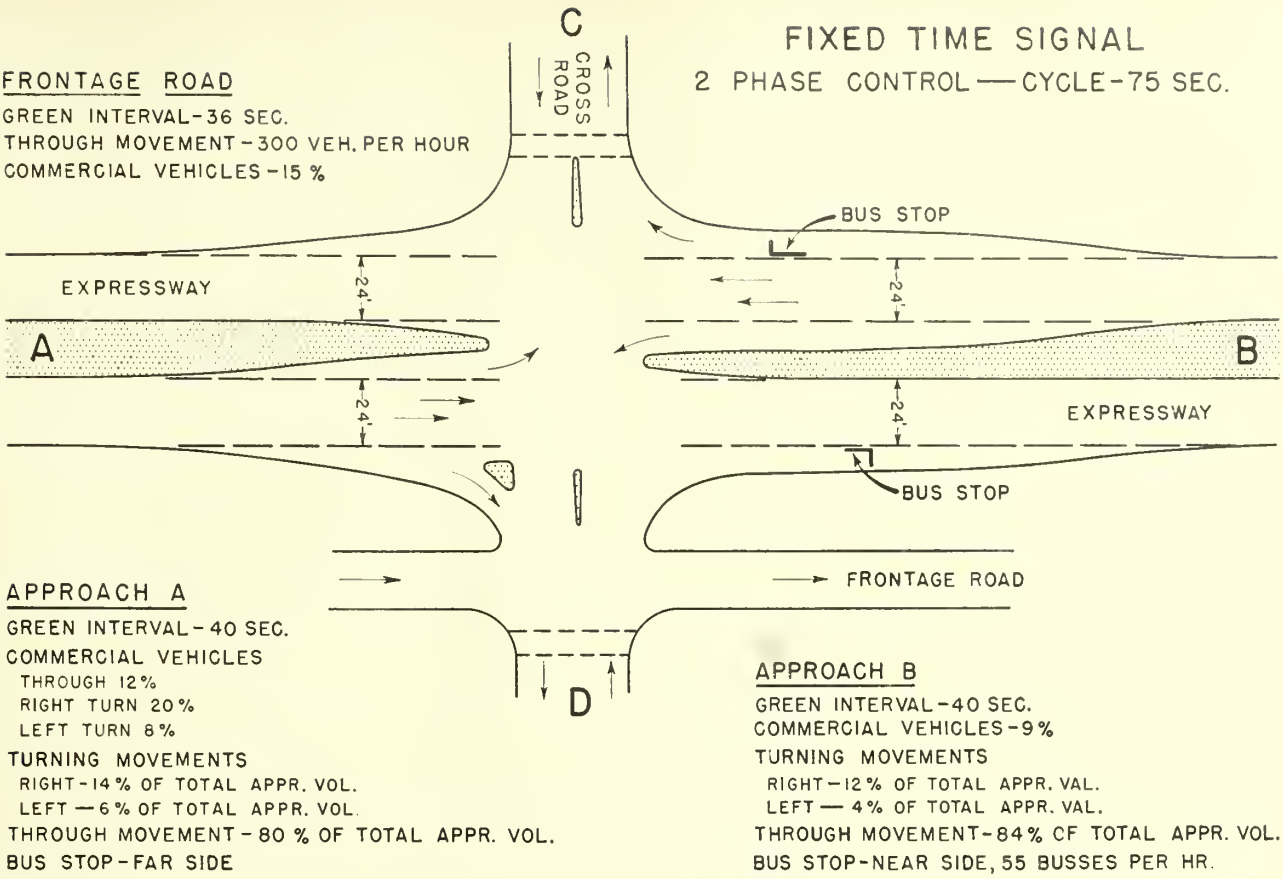


Figure 35.—Illustrative example 11.

Solution

APPROACH *A*

Through movement.— $1{,}000\times\frac{12}{10}\times 2=2{,}400$ passenger cars per hour of green (see adjustment V). With 12 percent commercial vehicles, $2{,}400\times 0.88=2{,}110$ vehicles per hour of green; or $2{,}110\times\frac{40}{75}=1{,}125$ vehicles per hour.

Right-turn movement.— $2{,}110\times\frac{14\%}{80\%}=370$ vehicles per hour of green, or $370-(0.20\times 370)+2\times 0.20\times 370=445$ equivalent passenger cars per hour of green.

The capacity of the right-turn lane: Volume on frontage road = 300 vehicles per hour, or $300\times\frac{75}{36}=625$ vehicles per hour of green, or $625+(0.15\times 625)=720$ equivalent passenger cars per hour of green.

Right-turn capacity = 1,200 − 720 = 480 passenger cars per hour of green, which is greater than the volume above.

Right-turn movement $=370\times\frac{40}{75}=200$ vehicles per hour.

Left-turn movement.— $2{,}110\times\frac{6\%}{80\%}=160$ vehicles per hour of green, or $160\times\frac{40}{75}=85$ vehicles per hour.

Capacity of left-turn lane (not less than two vehicles per cycle) $=2\times\frac{3{,}600}{75}=96$ vehicles per hour.

(NOTE.—The added turning lane should have sufficient storage capacity to accommodate four standing vehicles clear of the through lanes.)

Total capacity of approach A.— 1,125 + 200 + 85 = 1,410 vehicles per hour.

APPROACH *B*

Through movement.—

Adjustments:

Cause	*Effect*	*Factor*
Bus stop	−(12×½) = −6%	0.94
(See adjustment V-3-B-2)		
Commercial vehicles	−(9×1) = −9%	0.91
Total factor = 0.94×0.91 =		0.855

Practical capacity, less busses $=2{,}400\times 0.855\times\frac{40}{75}=1{,}095$ vehicles per hour.

Total practical capacity = 1,095 + 55 busses = 1,150 vehicles per hour.

Right-turn movement.— $1{,}150\times\frac{12\%}{84\%}=165$ vehicles per hour.

Left-turn movement.— $1{,}150\times\frac{4\%}{84\%}=55$ vehicles per hour.

Total capacity of approach B.— 1,150 + 165 + 55 = 1,270 vehicles per hour.

Example 12

Problem

An urban expressway in rolling terrain has two 12-foot lanes in each direction, and all cross streets are separated in grade except at one isolated intersection. At this intersection, calculations indicate that the volume of traffic on the cross street can be accommodated by a two-phase traffic signal if 30 percent of the elapsed time is allowed for movement on the cross street.

The problem is to determine the minimum number of lanes in each direction on the expressway, at the intersection, that will enable the intersection approaches to accommodate a volume of traffic equal to the capacity of the two 12-foot lanes where flow is uninterrupted. Turning movements from the expressway in either direction are: Right turns, 14 percent of approach volume; left turns, 5 percent of approach volume (see fig. 36).

Solution

A 65-second signal cycle is assumed. (Any other length of cycle, within reasonable limits, would be equally satisfactory provided that the shortest green interval is at least 20 seconds.) For the cross street, the green time will be 30 percent of 65, or 19.5 seconds, which may be rounded to 20 seconds. Allowing 5 seconds for amber periods, there remain 40 seconds of each cycle for go time on the expressway.

The practical capacity of the expressway under free-flowing conditions, with the prevailing number of commercial vehicles and in consideration of the nature of the terrain, is calculated as 1,100 mixed vehicles per lane per hour, or a total of 2,200 vehicles in each direction.

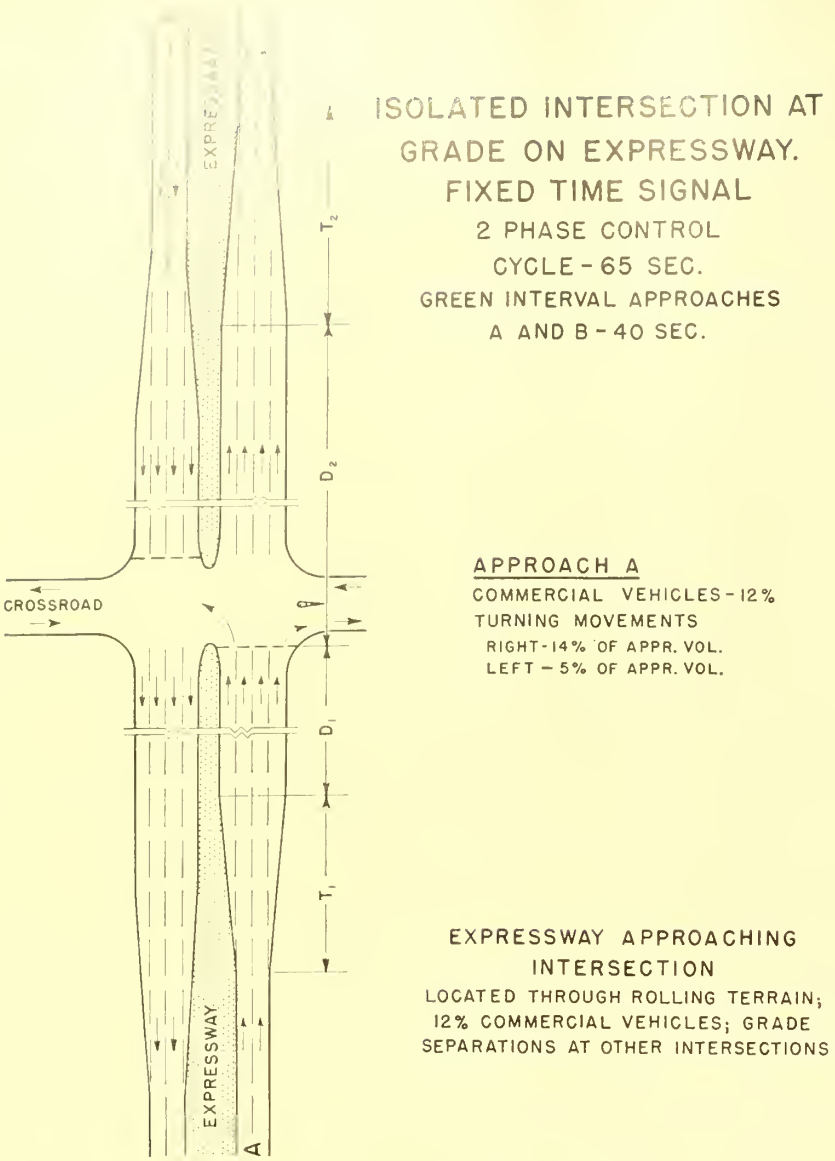


Figure 36.—Illustrative example 12.

The capacity per 12-foot lane at the intersection is 1,200 vehicles per hour of green less necessary adjustments. It should be noted that adjustments for turning movements are necessary (½ percent for right turns and 1 percent for left turns) because the lanes used by turning vehicles are also used by through traffic.

Adjustments:

Cause	*Effect*	*Factor*
Commercial vehicles	12×1 = −12%	0.88
Right turns	14×½ = − 7%	0.93
Left turns	5×1 = − 5%	0.95
Total factor = 0.88×0.93×0.95 =		0.78

Intersection capacity per lane (average) = $1{,}200 \times 0.78 \times \frac{40}{65} = 575$ vehicles per hour.

Number of lanes required in each direction = $\frac{2{,}200}{575} = 3.8$ lanes.

Four lanes, therefore, will be required in each direction at the intersection to accommodate a volume equal to the uninterrupted capacity flow of the expressway. It will be necessary that all lanes be continued through and beyond the intersection in the manner shown in figure 36 because the added lanes would be used by through traffic as well as by turning traffic.

The length of each added lane in advance of the intersection, in feet, (D_1 in fig. 36) should be not less than five times the green interval, in seconds, plus a gradual taper (T_1 in fig. 36). The combined length $D_1 + T_1$ should be sufficient to permit vehicles to decelerate to a complete stop from a normal operating speed on the expressway within this length.

The length of each added lane beyond the intersection (D_2 in fig. 36) should be one and one-half times the distance D_1, and the combined length $D_2 + T_2$ should be sufficiently great to permit vehicles to accelerate from a standstill to the operating speed of traffic on the expressway.

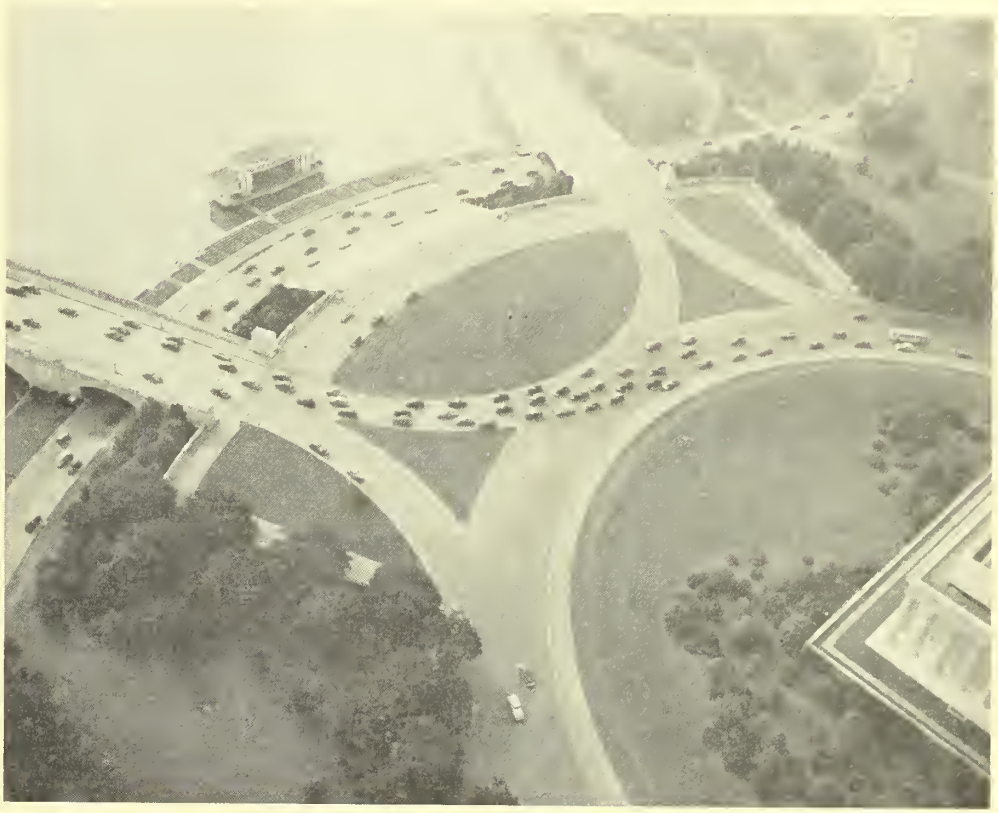
Part VI.—Weaving Sections; Unsignalized Cross Movements

Introduction

Weaving sections are provided for one purpose—to permit the crossing at grade of vehicle pathways with the least possible interference between vehicles. Weaving sections are usually selected as a compromise between the conventional intersection at grade, where delays are often excessive, and the grade separation, with its costly structure and appurtenances. Weaving sections are often provided as adjuncts to grade separations. The traffic circle is, in actuality, a series of weaving sections, and there are many other applications of the principle in the lay-out of controlled-access highways. **Unless consideration is given to that volume of traffic which must cross the path of other vehicles in reaching its destination, the capacity of these higher type facilities may very easily be overestimated.**

Two Classes of Traffic

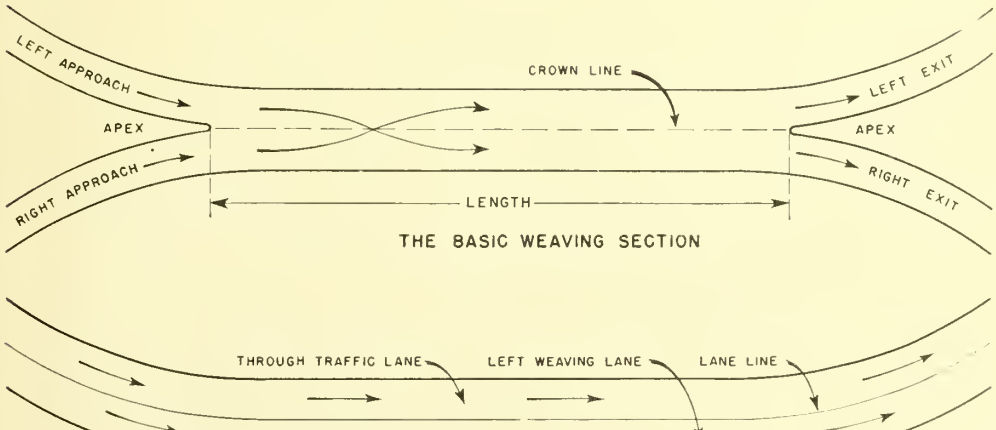
The vehicles using a weaving section fall logically into two classes: (1) Those entering, passing through, and leaving the section without crossing the normal path of other vehicles, and (2) those that must cross the paths of other vehicles after entering the section. The latter group are the weaving vehicles that make the facility necessary. Consideration of both types of traffic is essential in a study of capacity, but an understanding of the capacity of a weaving section is simplified if the behavior of each class of traffic is dealt with separately. On a well-designed facility operating below capacity, the two classes will actually separate themselves one from the other almost as positively as they do in theory.



A portion of a traffic circle, showing weaving sections. Rerouting of traffic on some of the approaches to this circle has been resorted to as a means of reducing the conflicting cross movement at this point.

Behavior of Weaving Vehicles

If all the vehicles entering a weaving section from either approach are destined to cross the path of all vehicles entering from the other approach—that is, if all traffic is weaving traffic—it is obvious that every car must cross the crown line somewhere between its extremities (see fig. 37). **At no instant can the number of vehicles in the act of crossing this crown line exceed the number than can crowd into a single lane;** provided, of course, that the facility is operating as it should without vehicles being required to come to a stop before merging with the stream of traffic from the other approach. **Thus, the total number of vehicles entering the weaving section, if all are weaving vehicles, cannot exceed the capacity of a single lane.** This, simply stated, is one of the rules governing the capacity of a weaving section. There are certain modifications of this rule, as will be brought out later, but for the sake of clarity discussion of these modifications is momentarily deferred.



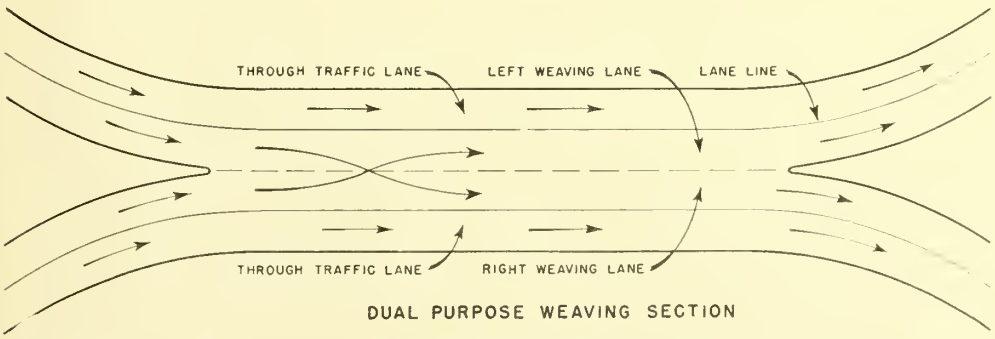


Figure 37 (above).—*The basic weaving section: all traffic weaves.*
Figure 38 (below).—*Dual purpose weaving section: serves weaving and nonweaving traffic.*

Speed Limited on Short Sections

One of the elements affecting the capacity of a traffic lane is the speed at which vehicles can or do travel. In the same manner, speed also influences the capacity of a weaving section. It is in this respect that the length of the section plays a part in its capacity. To understand the relation between speed and length of section, let it be imagined for the moment that we have a weaving section of very short length, say 50 or 100 feet.

Weaving sections on the Pentagon Network, Arlington, Va.

Further, let it be assumed that traffic is composed entirely of weaving vehicles. At very low traffic volumes there will be little conflict between weaving vehicles even on this short section because gaps in the stream of traffic from one approach will, in most cases, coincide with the entry of vehicles from the other approach. As traffic becomes heavier, however, the probability of vehicles entering the section from the two approaches simultaneously increases until at moderately heavy volumes many drivers will be required to stop and wait for a gap in the other stream of traffic. When the facility is taxed to its capacity, most vehicles will be required to come to a halt and the weaving section fails to serve its intended purpose. Operation is then comparable to that of an ordinary oblique intersection, having a capacity of about 1,200 vehicles per hour. This value corresponds to the possible capacity of a single traffic lane at low speeds.

Under more favorable conditions a weaving section of ample length may have a possible capacity that includes about 1,500 passenger cars per hour performing weaving maneuvers at an average operating speed of about 40 miles per hour. The needed length for this speed is about 900 feet. The same volume of weaving traffic can be accommodated at a speed of 30 miles per hour by a section 450 feet long.

Nonweaving Traffic

Only in rare instances will all traffic be weaving traffic although, as stated earlier, it is for service to the weaving traffic that the weaving section owes its existence. As a secondary purpose, **weaving sections must accommodate the nonweaving traffic by means of added lanes adjoining either side of the weaving lanes** (fig. 38). Determining the capacity of these nonweaving lanes involves no new principle, as the procedure is not different from that for any traffic lane on a multilane facility. **If any weaving section is to function properly and efficiently, it is important that these added lanes have adequate capacity to serve the nonweaving vehicles.** If nonweaving vehicles utilize the weaving lanes either through choice or through necessity, they interfere with the vehicles that must weave to reach their destination, thus reducing the total number of weaving vehicles that can be accommodated. Appropriate use of signs to direct drivers to the proper side of the approach road is essential, therefore, if the section is to operate efficiently during peak volume periods. Also, the effective length of a weaving section is influenced to some extent by the distance ahead of the weaving section that drivers on one approach road can see traffic on the other approach road. This distance may be used by drivers that must cross the paths of other vehicles to adjust their speeds before reaching the weaving section, so that the merging operation will be performed with a minimum of conflict between vehicles.

Typical Examples

Information regarding the relations between traffic volume, operating speed, and geometric features of weaving sections has been obtained from detailed studies conducted by the Bureau of Public Roads and from traffic volume data submitted by State highway departments and Committee members. Table 20 shows data pertinent only to the sections of considerable length which were operating at or near capacity and for which detailed data such as speeds are available.

Location No. 1—Pentagon Network

Weaving section No. 1 lies between two ramps and serves a part of the traffic to and from Arlington Memorial Bridge at Washington, D. C., as well as some traffic from Lee Boulevard destined for the Pentagon Building in Arlington, Va. Trucks are not allowed on this facility and, except for a negligible number of busses, traffic is composed entirely of passenger cars. There are no lane lines or longitudinal joints on the bituminous surface, but both approaches and the exit to the left, which serves as a ramp approaching the bridge, are used as two lanes. The weaving section is of sufficient width to accommodate four lanes of traffic. Figure 39 shows the number of vehicles in each lane on the approach roadways that entered each lane on the exit roadways but does not show the exact points at which the weaving maneuvers took place.

Vehicles from Lee Boulevard, or those approaching the weaving section from the right, traveled at an average speed of about 25 miles per hour. Speeds on the ramp from Memorial Bridge averaged 20 miles per hour with about 5 percent of the vehicles coming to a complete

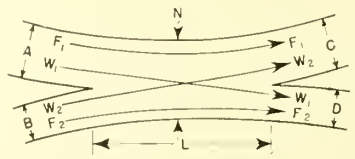


Table 20.—*Observed volumes on weaving sections*

Location number [1]	Dimensions						Traffic volume in vehicles per hour						Approximate speeds
	Length L	Number of lanes at—					W_1	W_2	W_1+W_2	F_1	F_2	Total	
		A	B	N	C	D							
	Feet												*M. p. h.*
1	1,140	2	3	4	2	3	1,496	1,794	3,290	0	246	3,536	25
2	1,146	2	3	[2] 4	2	2	772	1,712	2,484	0	240	2,724	18
3	921	3	3	4	3	2	1,000	309	1,309	915	790	3,014	40
4	488	3	2	3	2	3	1,698	276	1,974	450	348	2,772	22
5	550	3	3	5	2	3	419	1,676	2,095	342	764	3,201	28
5	550	3	3	5	2	3	1,351	384	1,735	450	582	2,667	30
6	550	3	3	5	2	3	945	755	1,690	714	437	2,841	27
6	550	3	3	5	2	3	509	1,672	2,181	638	425	3,244	25

[1] Locations 1–4 are on the Pentagon Network, Arlington, Va.; locations 5 and 6 are on the San Francisco Bay Bridge distribution structure.
[2] Narrows from 44 feet to 27 feet approximately midway of length.

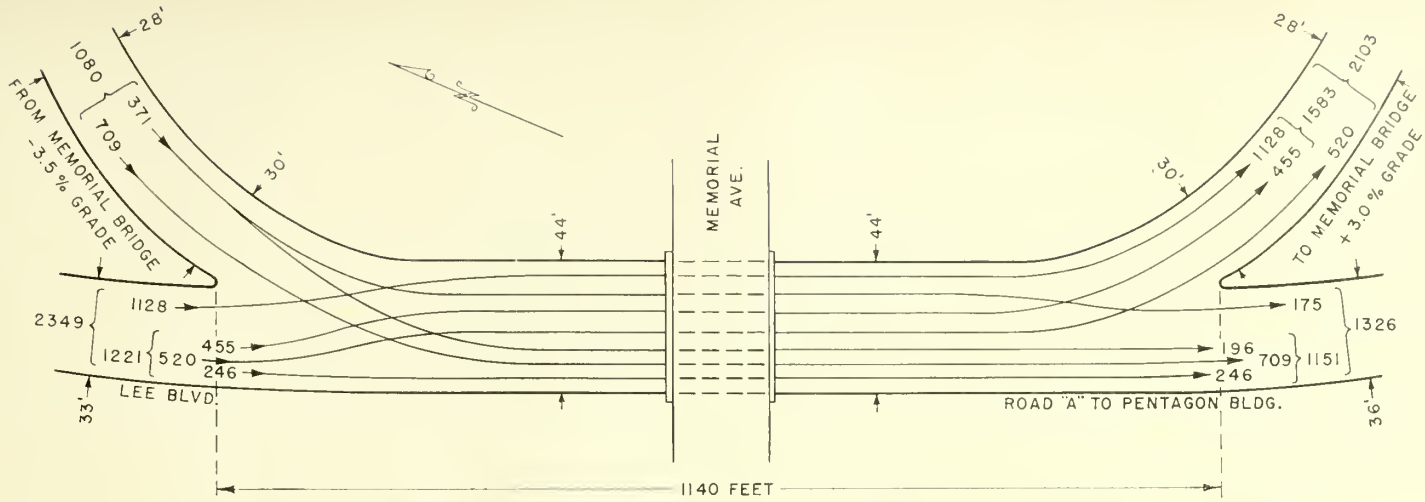


Figure 39.—Weaving section on roadway under Memorial Avenue near west end of Memorial Bridge, Washington, D. C. (morning rush period, 7:45–8:45 a. m., November 13, 1947).

stop before entering the weaving section. Near the exit from the weaving section, vehicles in the left lane were spaced as closely together as possible and all were moving at practically the same speed—15 to 20 miles per hour. The average speed in the right-hand lane near the exit was about 25 miles per hour with 20 percent of the vehicles traveling 30 to 35 miles per hour.

It was apparent during the period of peak volume that the desired number of weaving maneuvers could not be performed in a safe and efficient manner, as evidenced by the 175 vehicles destined to the right-hand exit that were forced to stay in the left lane for the full length of the weaving section before crossing the path of vehicles taking the left-hand exit. Also, it was obviously impossible for all vehicles turning left to Memorial Bridge to enter the left lane, which accommodated 1,583 vehicles in the 1 hour and frequently accommodated vehicles at a rate approaching the basic capacity of 2,000 vehicles per hour for periods of several minutes duration.

Location No. 2—Pentagon Network

Figure 40 shows the details of location No. 2, the same facility as location No. 1, but as it appeared in 1946 prior to improvement to the present lay-out. It will be noted that the improved facility (fig. 39) accommodates about 700 vehicles per hour more than the number observed on its predecessor. This over-all increase consists almost entirely of weaving vehicles. The service rendered to 3,429 vehicles is comparable to that previously rendered to the 2,724 vehicles per hour using the former facility. Since the improvement, there has been a slight increase in speed, and weaving maneuvers can be performed with somewhat greater ease. The present facility provides slightly better service because only 175 vehicles, as compared with 270 on the earlier facility, are forced to travel the full length of the section in the left lane before being able to cross to the exit leading to road A. On the old facility, represented by figure 40, nearly all weaving maneuvers were performed on that portion of the section north of Memorial Avenue, whereas on the newer facility, weaving is accomplished throughout the length of the weaving section.

Location No. 3—Pentagon Network

Pertinent physical features and the maximum observed traffic volume at location No. 3 on the Pentagon Network, commonly referred to as the southbound mixing lanes, are shown in figure 41. At this location the two approach routes and the two exit routes are of equal importance so no one of the four traffic movements has been shown any preference. The arrows show only the total volume entering and leaving on each roadway.

Studies have shown that the average speed is 37 miles per hour on the two approach roads, with the following distribution of speeds:

	Percent
Below 30 m.p.h.	14
30 to 39 m.p.h.	53
40 to 49 m.p.h.	30
50 m. p. h. or over	3

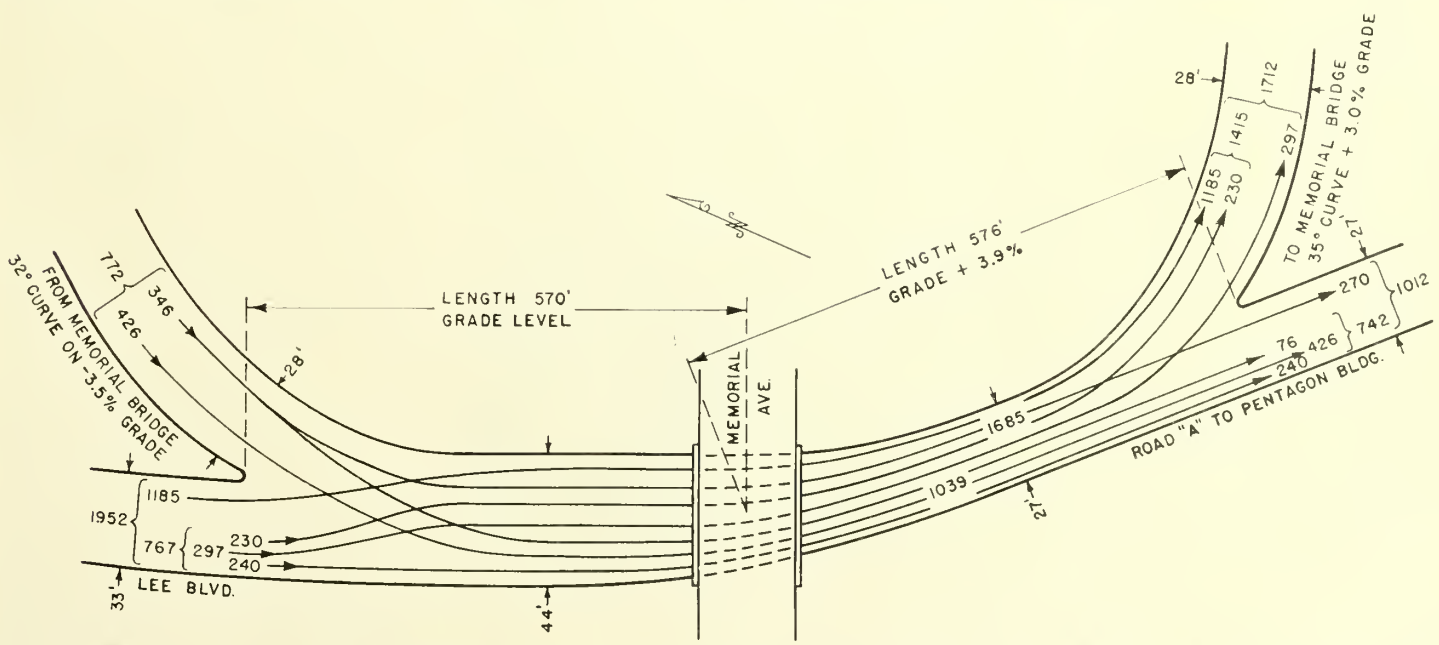


Figure 40.—Maximum observed hourly traffic volume on mixing lane section on roadway under Memorial Avenue near west end of Memorial Bridge, Washington, D. C.

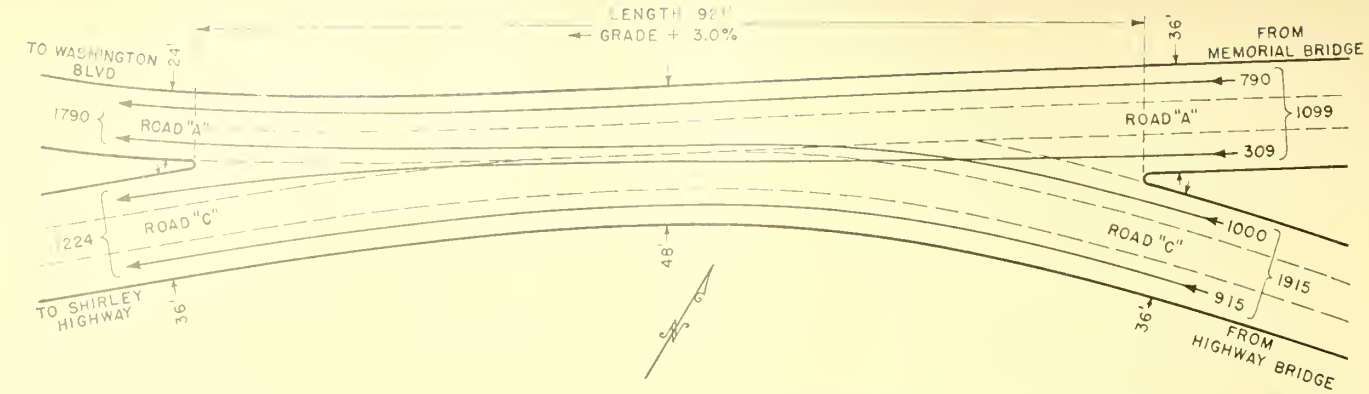


Figure 41.—Maximum observed hourly traffic volume on mixing lanes for south-bound traffic at junction of road A from Memorial Bridge and road C from U S 1, Arlington, Va.

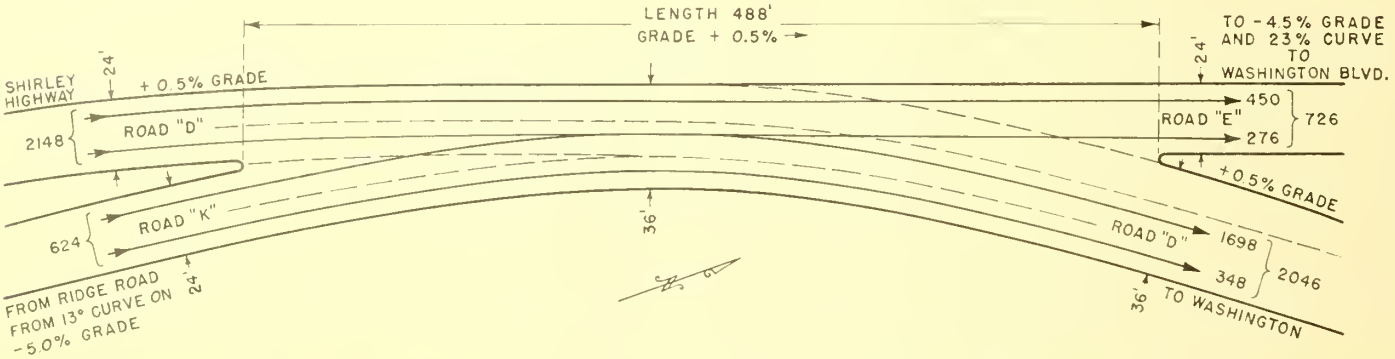


Figure 42.—Maximum observed hourly traffic volume on mixing lanes for north-bound traffic at junction of road K from Ridge Road and the Shirley Highway, Arlington, Va.

Very few drivers reduce their speeds while on or approaching the weaving section during off-peak periods. During the peak period represented in figure 41, however, there was a very noticeable difference between the average speeds on the approach roads and the speeds on the weaving section. Few vehicles involved in the weaving maneuvers traveled at speeds in excess of 30 miles per hour, and on several occasions during the hour the two weaving lanes became filled for the entire length of the section with vehicles at a standstill. These periods generally lasted for less than a minute and the condition usually developed when a queue of vehicles from each of the two approach roads traveled nearly the entire length of the section at approach road speeds without performing the necessary weaving maneuvers before coming to a complete stop immediately ahead of the apex separating the exit roadways.

During the hour of study, the total flow was 3,014 vehicles, including 309 passenger cars from one approach road that crossed the paths of 1,000 vehicles (4 percent dual-tired trucks and busses) from the other approach road. Although the roadway appeared to have reached its practical capacity, the volume of traffic approaching on the right and leaving on the right, or approaching on the left and leaving on the left, could have been considerably greater without increasing the congestion had these vehicles not become involved with the ones that were weaving. Also, lower approach-road speeds would have permitted a larger number of vehicles to perform the necessary weaving maneuvers within the same length of weaving section without increasing the apparent congestion. This section exceeds the minimum length required for the prevailing volume of traffic, but the added length overcomes in part the deficiency of inadequate signs and the absence of lane lines.

Entrance turn from ramp to freeway. Although traffic from the ramp is rather light, it is sufficient to fill the available spaces in the first lane of the freeway. The lane adjacent to the median is almost devoid of traffic. This characteristic tendency for traffic to distribute itself between lanes results in many of the spaces being shielded against occupancy by vehicles from the ramp.

Location No. 4—Pentagon Network

The traffic movements at location No. 4 on the Pentagon Network are shown by figure 42. In this case, traffic approaching on the Shirley Highway (road *D*) headed toward Washington (also road *D*) has been shown a definite preference by the arrangement of the longitudinal joints, which act as lane lines,

and by the general alinement. Traffic slowed to about 20 miles per hour for short periods during the hour of peak volume. A slight increase in the total volume of weaving traffic would have resulted in very congested conditions.

Weaving Sections Limited in Practical Application

The results of analysis of available data on traffic volumes and speeds on weaving sections are shown by figure 43. Basically, traffic on a weaving section is affected by density in much the same manner as on a roadway with uninterrupted flow. Maximum volumes occur at speeds between 20 and 30 miles per hour. Higher speeds are possible only at volumes and traffic densities lower than those found when the facility is operating at its possible capacity. **Whenever traffic density exceeds the critical density, speeds fall below 20 miles per hour, the capacity is lowered, and complete congestion or stagnation may occur within a few seconds.**

Weaving section for north-bound traffic at the junction of Shirley Highway and Washington Boulevard, Arlington, Va.

The curves in figure 43 show that there is a rapid increase in the length of the section required for a given speed with an increase in the number of weaving vehicles. Doubling the traffic volume approximately triples the length of section required and doubles the number of lanes required for the weaving vehicles.

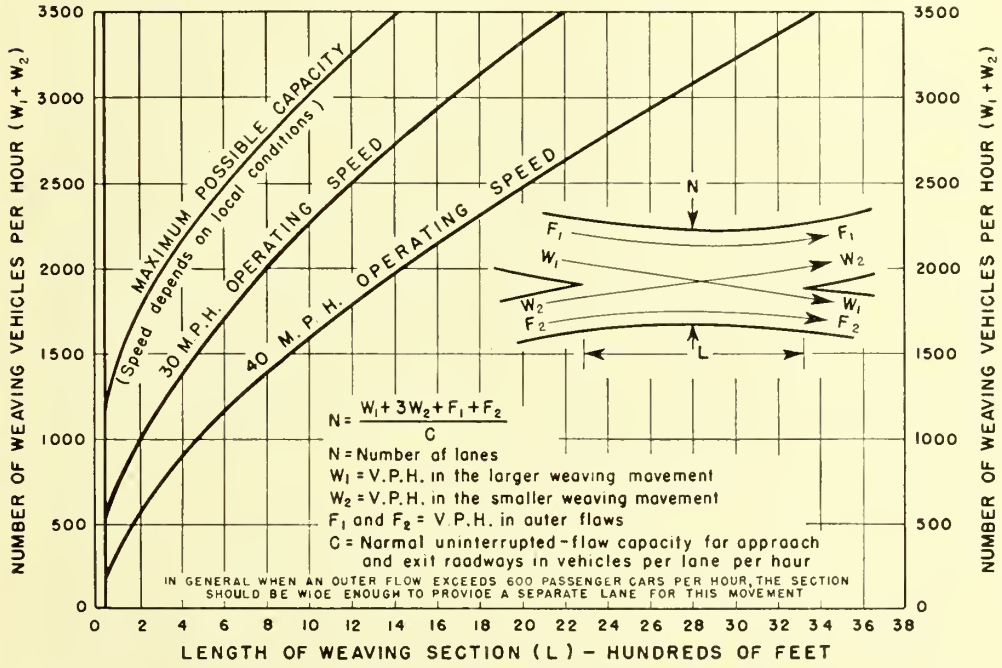

NUMBER OF WEAVING VEHICLES PER HOUR (W_1+W_2)
LENGTH OF WEAVING SECTION (L) – HUNDREDS OF FEET
MAXIMUM POSSIBLE CAPACITY
(Speed depends on local conditions)
30 M.P.H. OPERATING SPEED
40 M.P.H. OPERATING SPEED
$N = \frac{W_1+3W_2+F_1+F_2}{C}$
N = Number of lanes
W_1 = V.P.H. in the larger weaving movement
W_2 = V.P.H. in the smaller weaving movement
F_1 and F_2 = V.P.H. in outer flows
C = Normal uninterrupted-flow capacity for approach and exit roadways in vehicles per lane per hour
IN GENERAL WHEN AN OUTER FLOW EXCEEDS 600 PASSENGER CARS PER HOUR, THE SECTION SHOULD BE WIDE ENOUGH TO PROVIDE A SEPARATE LANE FOR THIS MOVEMENT
N
L

Figure 43.—Operating characteristics of weaving sections.

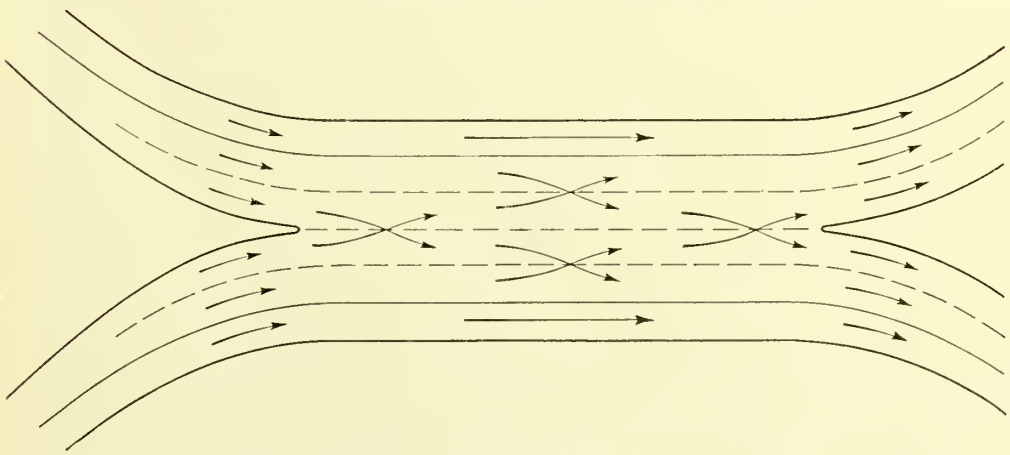


Figure 44.—A compound weaving section.

Figure 44 shows schematically the weaving maneuvers that must take place when the number of weaving vehicles is double the normal capacity of a traffic lane. All vehicles are shown crossing the crown line either in the first one-third or last one-third of the section. Each vehicle is involved in two weaving maneuvers with the result that four times as many weaving maneuvers must be performed as with half the volume. Theoretically, at least, this illustrates the need for tripling the length for twice the volume.

In practice, however, most drivers, knowing in advance that they must cross the crown line, attempt to gain a position on the approach road which is most favorable for the early performance of the weaving maneuver. As a result, the two streams of vehicles that must weave approach each other with no gaps to permit an interchange of positions. **Any weaving section, regardless of its length or number of lanes, will become badly congested when the number of weaving vehicles approaches the possible capacity of two traffic lanes. Operating conditions will seldom be entirely satisfactory unless the traffic on the approach roadways is well below the practical capacities of these approaches and the weaving section has one more lane than would normally be required for the combined traffic from both approaches. For this reason, weaving sections are considered practical only where the two intersecting one-way roadways each carries less than the normal capacity of two lanes of a one-way roadway and the total number of vehicles required to weave does not exceed 1,500 per hour.** In computing capacities, allowance must be made for trucks, lanes of substandard width, etc., on the same basis as in the case of multilane roadways.

Part VII.—Ramps and Their Terminals

INTRODUCTION

The efficiency of traffic movement on freeways or expressways and the extent to which their potential capacities can be realized depends directly on the adequacy of the facilities that are provided for entering and leaving these highways. Improperly planned entrances can seriously limit the traffic volumes that can use an expressway, and exit facilities incapable of accommodating vehicles leaving the highway at one point, even though the number be relatively few, can cause complete congestion of all traffic.

While the subject of ramp capacities has been considered by the Committee and much factual data have been collected, only a few generalizations can be drawn at the present time that might be helpful in dealing with problems concerning ramp capacities. No exact rules are applicable to all ramps because their capacities are so closely related to the particular lay-out, especially at the terminals. The primary purpose of this section, therefore, is to furnish some knowledge of traffic operations on ramps that can be utilized by engineers in rationalizing the various elements that should be considered.

The capacity of a ramp is affected by the character of traffic, gradient, width, curvature, and the speed at which vehicles operate. **The sharp curvature on most ramps usually limits the possible capacity to that attained on a tangent section at speeds below 20 miles per hour.** At the higher speeds, drivers find it difficult to stay within their lane and have a tendency to maintain a somewhat greater headway than is normal on tangent sections. This happens despite the fact that lanes wider than 12 feet are often provided on ramps.

Except on ramps having extremely long radii, and on direct connections having a low degree of curvature, the lane capacity of the ramp itself, when the entrance or exit does not govern, is usually of the order of 1,200 passenger cars per hour. Thus, a ramp having a nominal width of two lanes (usually 28 or 30 feet) should accommodate about 2,400 passenger cars at an average speed of 12 to 15 miles per hour. Actual examples of ramp movements of this magnitude are rare, indeed, because few of the terminals are so planned that traffic entering and leaving the ramp can do so with the necessary freedom of interference from traffic on the major highways or streets.

Even on some of the most modern facilities, there is a tendency for traffic to move in a single line at one or two points on the ramp. The point of constriction is usually at either the entrance or the exit to the ramp. Under such conditions the possible capacity of the ramp will be about 1,200 vehicles per hour. This probably accounts for the common belief that the capacity of any ramp will not exceed 1,200 to 1,500 vehicles per hour and therefore the ramp should be tapered to confine vehicles on the ramp to one lane as they enter an expressway. **However, there is little reason to think that ramp terminals cannot be so planned that much larger volumes of traffic can be handled with ease.** Further, there are a few existing facilities where much higher volumes have been observed. One such example is the ramp shown as the left exit of the weaving section illustrated in figure 39. Nearly every day about 2,100 vehicles per hour during the morning rush hour use this ramp, which is approximately semicircular in shape and has a radius slightly over 200 feet. The vehicles from the ramp merge with about 1,200 vehicles per hour in three lanes at the ramp terminal.

Another example of an unusually heavily traveled ramp is the Dearborn Street interchange on North Lakeshore Drive in Chicago. At the on-bound ramp 2,092 vehicles in 1 hour have been observed merging with 5,923 vehicles on the expressway. The ramp is a direct connection and traffic merges in six lanes. The off-bound ramp at this interchange has been observed to discharge 2,482 vehicles in 1 hour. This is also a direct connection.

Where direct connections having sufficiently large radii of curvature and superelevations to permit speeds of 30 miles per hour or more during heavy traffic volumes are employed, the practical capacities of the lanes on the ramp and the expressway may be calculated as for any traffic lane in the manner outlined in part IV, providing traffic using the ramp is not hampered in entering and leaving the ramp by through traffic on the highways which the ramp connects. Generally, however, the capacity of a ramp is controlled by conditions at or near the terminals.

Ramps may be of different forms, but the two general classes are inner loops and direct connections.

CONDITIONS AFFECTING RAMP CAPACITY

There are almost an unlimited number of different conditions and combinations of conditions that limit the number of vehicles that can use a given ramp, or that determine the necessary ramp design for a certain number of vehicles. Only a few of the more important conditions that must be considered are covered

in the following discussion. Any one of these might be the controlling factor for a particular location.

Volumes During Peak Periods

For no portion of a highway is it more important to know the volume of traffic in the various movements during peak periods than when trying to design an adequate ramp or when estimating the capacity of an existing ramp. Annual volumes or 24-hour volumes for the various movements are of little value, because the peak volumes on some ramps do not coincide with the peak volumes on the highways. Oftentimes the peak volume on one ramp will occur during periods when the traffic volumes on other nearby ramps or on the highway are relatively low. For this reason, the traffic volume that can be accommodated by a particular ramp will vary from time to time depending on the traffic volumes using the facilities that it connects.

Weaving Distance Between Ramps

The number of vehicles that can leave or enter a ramp is sometimes controlled by the number of vehicles leaving or entering the same highway on adjacent ramps. In the case of the two inner loops of a cloverleaf design which are used by traffic traveling in the same direction on a main highway, traffic entering the main highway from the on-ramp must weave with traffic entering the off-ramp. For this condition, the capacity of the section of roadway between the two ramps can be calculated in the same manner as for any weaving section. **If the distance between the two inner loops is very short, the combined capacity of the two ramps would be restricted to a total of about 1,200 passenger cars per hour at a speed of less than 20 miles per hour.** This is one reason that designs in which vehicles leave an expressway facility ahead of the point where vehicles enter the facility are preferred to the cloverleaf design, especially when the movements on two adjacent inner loops are at or near their peaks at the same time.

Terminal Conditions

The entrance to a ramp might be made from a facility on which the flow of traffic is uninterrupted, or it might be from a highway or city street with cross traffic at grade and be in the immediate vicinity of an intersection controlled by traffic lights. Likewise, the exit from a ramp might be made either to a facility with uninterrupted flow or to a highway or street with cross traffic at grade. Furthermore, either or both of the roadways which the ramp connects might have acceleration and deceleration areas or added lanes for the vehicles using the ramp. Also, if there are no acceleration areas or added lanes, the general lay-out may be such as to require traffic to stop before entering the highway or street. **There are many different conditions that can exist at the entrance and exit of a ramp, any one of which might control the number of vehicles that can use the particular ramp.**

At-grade intersections in vicinity of ramp

Traffic approaching a ramp leading to an expressway and traffic on a ramp leading from an expressway must oftentimes pass through an intersection at grade in the immediate vicinity of the ramp. **The volume of traffic that can be accommodated by the ramp is then dependent on the capacity of the nearby intersection.** When this is the case, the information on intersection capacities should be applied when estimating the maximum volume of traffic that can enter or leave the ramp.

Entrance to expressway from ramp

It is a common practice to assume that as many vehicles can enter an expressway from a ramp as the difference between the capacity of the expressway beyond the ramp and the traffic volume on the expressway ahead of the ramp. For example, if a four-lane divided expressway had a practical capacity of 3,000 vehicles per hour in the one direction and there were 1,800 vehicles per hour on the expressway just ahead of the on-ramp, it would normally be assumed that 1,200 vehicles per hour could enter the expressway from the ramp. With no acceleration area, this would be true only if the 1,800 vehicles already on the expressway could and would crowd into the one traffic lane before reaching the on-ramp. It would also be true if there were an acceleration area of the same length as the distance necessary for 3,000 vehicles traveling in three lanes to crowd into two lanes without a marked reduction in speed.

Obviously, vehicles from a ramp can enter an expressway only in the available spaces between vehicles operating in the first lane of the expressway. **The higher the traffic volume on the expressway, the less opportunity there will be for vehicles to enter the expressway when other conditions are comparable.**

The length of the space between vehicles on the expressway that will be utilized by a driver entering from a ramp will vary for different drivers and with the frequency of such spaces and the speed of traffic. If a driver has to wait a long time for the occurrence of a space which he normally considers entirely adequate, he is apt to take advantage of a somewhat shorter space that occurs more frequently.

The results of a study made to determine the spacing between vehicles in the first lane of an expressway that drivers utilize in entering from a ramp on the right are shown by figure 45. These results represent only one particular set of conditions, i. e., a continuous back-log of vehicles on a ramp 20 feet wide with a stop sign strictly enforced and no acceleration area for traffic before entering the through lanes of the expressway. To eliminate or at least reduce the effect of one of the variables—the speed of traffic on the expressway—the spacing between vehicles on the expressway has been expressed as the time interval between vehicles as they approached the location of the ramp.

The upper part of figure 45 shows that some drivers, although very few, entered the expressway from the ramp when the time spacing (center to center) between passenger cars in the first lane of the expressway was as low as 1 second. Other drivers would not enter unless the time spacing was as high as 17 seconds. In other words, with a double line of vehicles on the ramp waiting to enter

A ramp terminal where the traffic demand exceeds the capacity of the expressway. During peak periods traffic must be directed by a police officer.

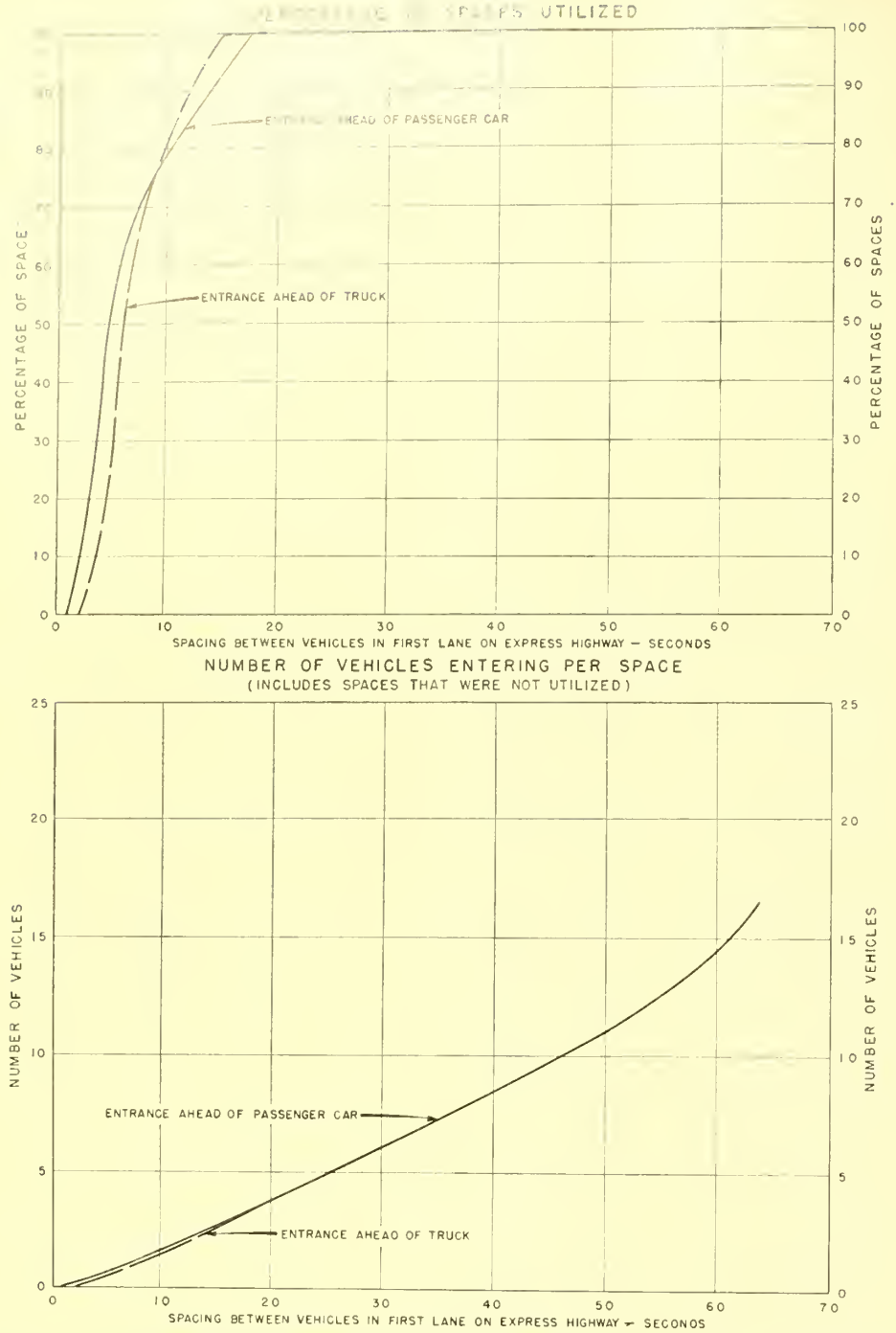

PERCENTAGE OF SPACES UTILIZED
ENTRANCE AHEAD OF PASSENGER CAR
ENTRANCE AHEAD OF TRUCK
PERCENTAGE OF SPACE
PERCENTAGE OF SPACES
SPACING BETWEEN VEHICLES IN FIRST LANE ON EXPRESS HIGHWAY — SECONDS
NUMBER OF VEHICLES ENTERING PER SPACE
(INCLUDES SPACES THAT WERE NOT UTILIZED)
NUMBER OF VEHICLES
ENTRANCE AHEAD OF PASSENGER CAR
ENTRANCE AHEAD OF TRUCK
SPACING BETWEEN VEHICLES IN FIRST LANE ON EXPRESS HIGHWAY — SECONDS

Figure 45.—Utilization of available spaces between vehicles in the first lane of an express highway by vehicles entering from a ramp 20 feet wide with stop control (during periods when two lines of vehicles were waiting to enter the expressway from the ramp).

the expressway, none of the spaces shorter than 1 second between vehicles in the first lane of the expressway were utilized by vehicles from the ramp. Likewise, 20 percent of the 3-second spaces, 54 percent of the 5-second spaces, and all of the spaces 17 seconds or longer were utilized by vehicles in entering the expressway from the ramp. When the space occurred ahead of a truck on the expressway, an entrance from the ramp was never made unless the time space was at least 2 seconds and at least one vehicle always entered when the time space was 15 seconds.

The lower part of figure 45 shows the average number of vehicles that entered the various spaces in the first lane of the expressway. Although only 54 percent of the 5-second spaces were utilized, an average of 2.8 vehicles entered each of the utilized spaces making an average of 1.5 vehicles that entered the expressway for each available 5-second space. The detailed data show that vehicles which utilized spaces less than 4 seconds long came almost entirely from the right-hand lane of the ramp, whereas approximately half of the vehicles that entered any space longer than 4 seconds came from the left lane of the ramp.

Lane usage

Information of the type shown by figure 45 in combination with lane usage on an expressway and the frequency of occurrence of various time spaces between vehicles at different hourly volumes may be used to determine the maximum number of vehicles that can enter an expressway from a ramp when there is no acceleration area. With uninterrupted flow, the frequency of occurrence of various time spaces between vehicles in a lane is very consistent for a given traffic volume regardless of the other prevailing conditions, and can be predicted with a high degree of accuracy from the results of comprehensive studies (see figs. 9 and 10 in part IV). Lane usage, or the percentage of vehicles in each lane, will vary, however, for different total traffic volumes and for a large number of physical and geometric conditions that might prevail in the vicinity of a ramp.

On a modern four-lane express highway at tangent locations that are not in the vicinity of an intersection, traffic will generally be distributed between the two lanes for the one direction of travel approximately as shown by figure 46. **As the traffic approaches an interchange or a ramp, the distribution between lanes will change and the extent of the change will depend on the particular lay-out at the interchange, the amount of traffic that enters and leaves the expressway at the interchange or ramp, and the effectiveness of the directional signs and pavement markings.**

Figure 47 shows the distribution of traffic between the lanes on an expressway at the approach to a ramp on the right with a continuous back-log of vehicles trying to enter the expressway. In the one case, as shown by the solid line, traffic from the ramp was required to stop before entering the expressway whereas, in the other case, as shown by the broken line, an acceleration lane was available and entering traffic did not necessarily stop.

For both conditions shown in figure 47, most of the traffic on the expressway used the second or left-hand lane during low traffic volumes to avoid interference with traffic entering from the ramps. **As the traffic volume on the expressway increased, however, the distribution between lanes became more nearly equal, there being a somewhat greater tendency for the drivers to avoid the right-hand lane when entering traffic was not required to stop than when it was required to stop. With an acceleration area available, therefore, it is not only easier to perform the merging operation than when there is a stop sign on the ramp and no acceleration area, but there are more spaces between vehicles in the first lane of the expressway that can be utilized by traffic entering from the ramp.**

The most important point illustrated by figure 47, however, is that **the effect which the ramp has on the distribution of traffic between lanes during low traffic volumes on the expressway is not apparent when the volume exceeds 1,000 vehicles per lane. Drivers will not crowd one lane to leave another lane comparatively free of traffic so that other drivers from a ramp can enter more**

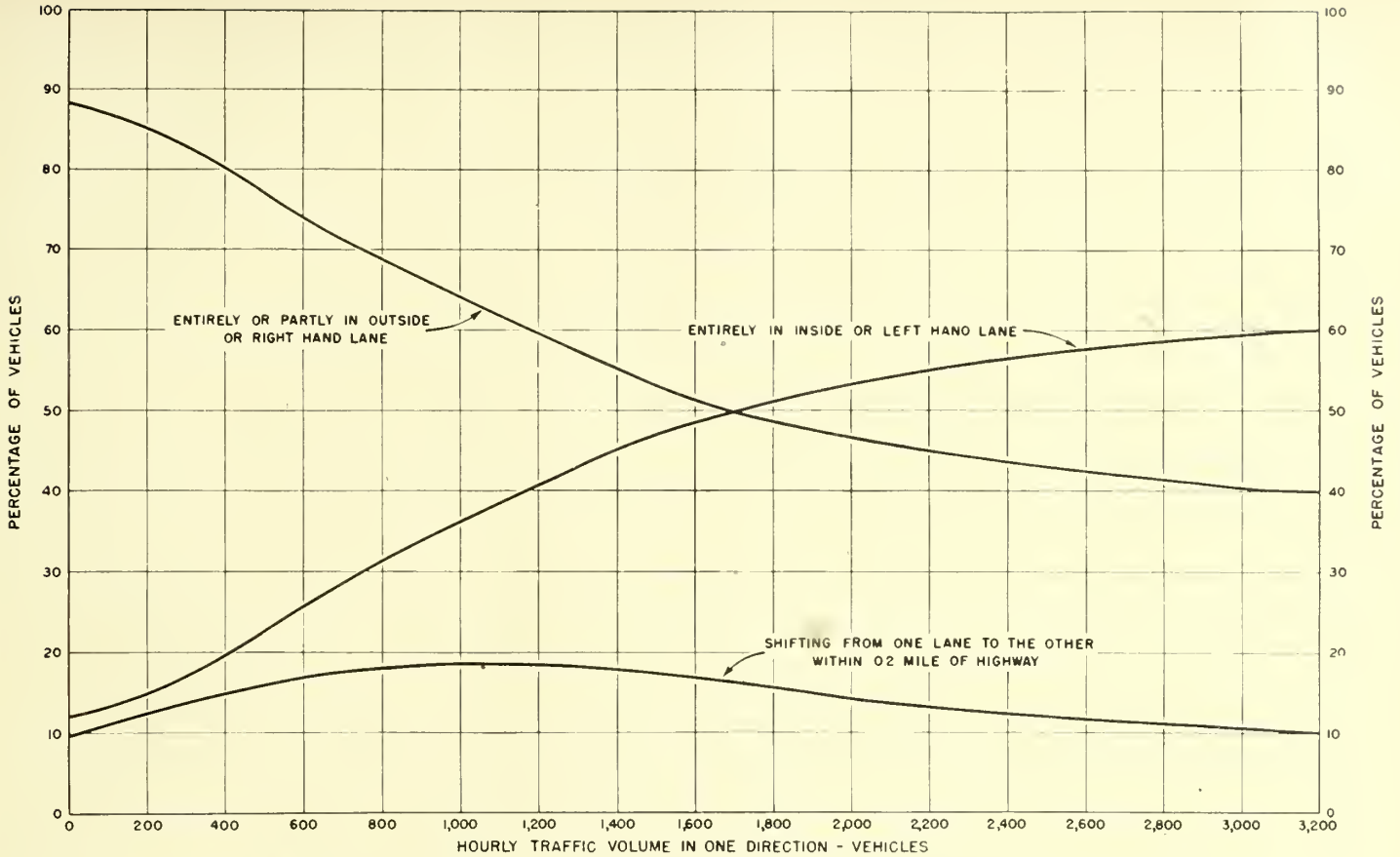

ENTIRELY OR PARTLY IN OUTSIDE OR RIGHT HAND LANE
ENTIRELY IN INSIDE OR LEFT HAND LANE
SHIFTING FROM ONE LANE TO THE OTHER WITHIN 02 MILE OF HIGHWAY
PERCENTAGE OF VEHICLES
HOURLY TRAFFIC VOLUME IN ONE DIRECTION - VEHICLES

Figure 46.—Distribution of vehicles between traffic lanes on a four-lane highway during various hourly traffic volumes.

easily. This is one reason that it is not proper to assume that as many vehicles can enter an expressway from a ramp as the difference between the capacity of the expressway beyond the ramp and the traffic volume on the expressway ahead of the ramp.

The possible capacity of a particular ramp in relation to the volume of traffic on an expressway is shown by figure 48. The design at this location provided a stop sign to control traffic from the ramp. Recently, however, police-officer control has been necessary during peak periods to handle the increased traffic volumes without long delays to traffic on the ramp. Stopping through traffic on a facility designed for freeway or expressway operation is extremely undesirable but, as in this case, such a procedure will eventually have to be resorted to on many of the freeway facilities now being designed unless careful consideration is given to capacity aspects of the designs employed in the vicinity of ramps. In at least one case, it has been necessary to prohibit the use of a ramp during peak periods to prevent traffic on the entire expressway from coming to a standstill.

The conditions illustrated by figure 48 represent an expressway with a maximum possible capacity of 3,050 vehicles per hour in the one direction and a ramp with a maximum possible capacity of 2,480 vehicles per hour. Neither of these capacities could be realized, however, except when the traffic volume on one of the two was zero. The greatest difference in total capacity between police-officer and stop-sign control occurred when the volume approaching on the expressway was about 1,500 vehicles per hour. Under this condition, officer control increased the total flow of vehicles from 2,130 per hour to 2,900 per hour, or 36 percent. Even with this increase, however, the traffic that could enter from the ramp was limited to 1,400 vehicles

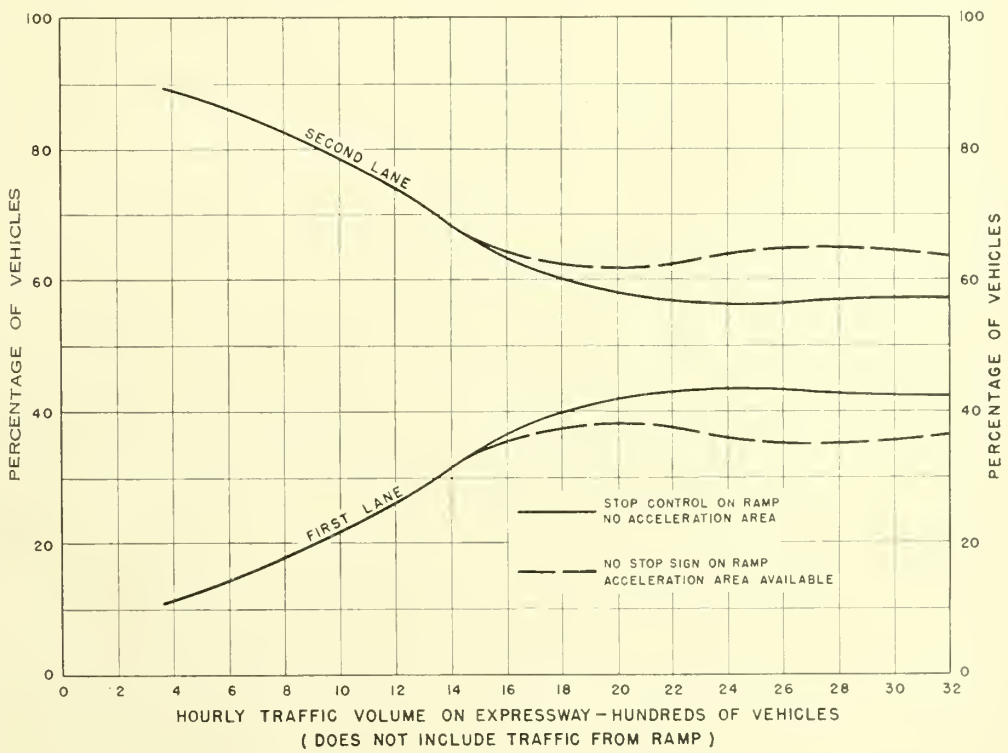
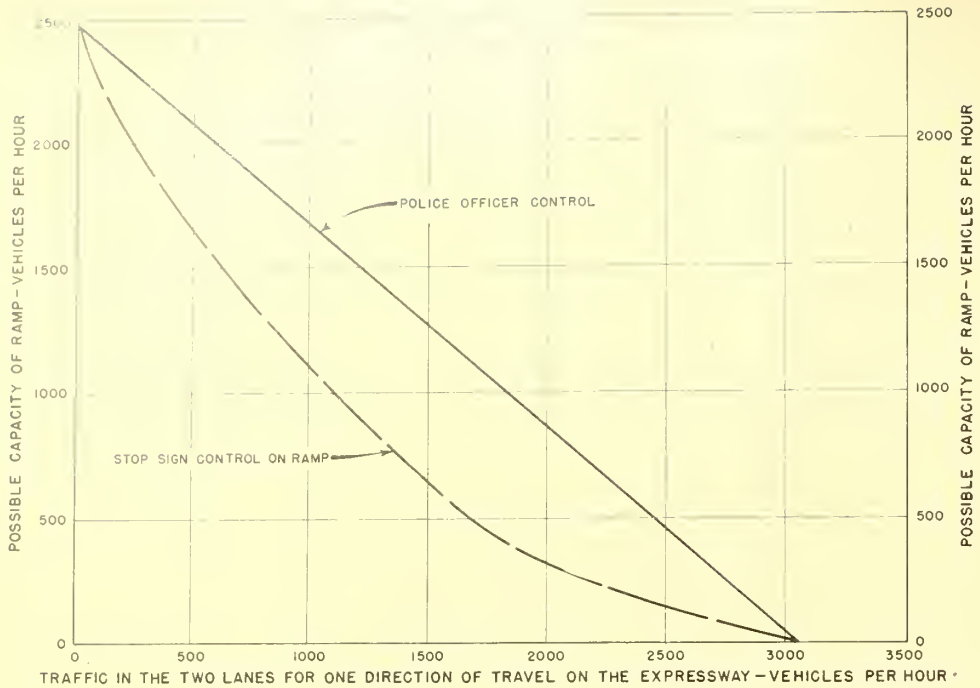

SECOND LANE
FIRST LANE
STOP CONTROL ON RAMP
NO ACCELERATION AREA
NO STOP SIGN ON RAMP
ACCELERATION AREA AVAILABLE
PERCENTAGE OF VEHICLES
HOURLY TRAFFIC VOLUME ON EXPRESSWAY - HUNDREDS OF VEHICLES
(DOES NOT INCLUDE TRAFFIC FROM RAMP)

Figure 47.—Distribution of vehicles between traffic lanes on a four-lane expressway at approach to ramp where heavy volume enters the expressway on the right.


POLICE OFFICER CONTROL
STOP SIGN CONTROL ON RAMP
POSSIBLE CAPACITY OF RAMP-VEHICLES PER HOUR
TRAFFIC IN THE TWO LANES FOR ONE DIRECTION OF TRAVEL ON THE EXPRESSWAY-VEHICLES PER HOUR

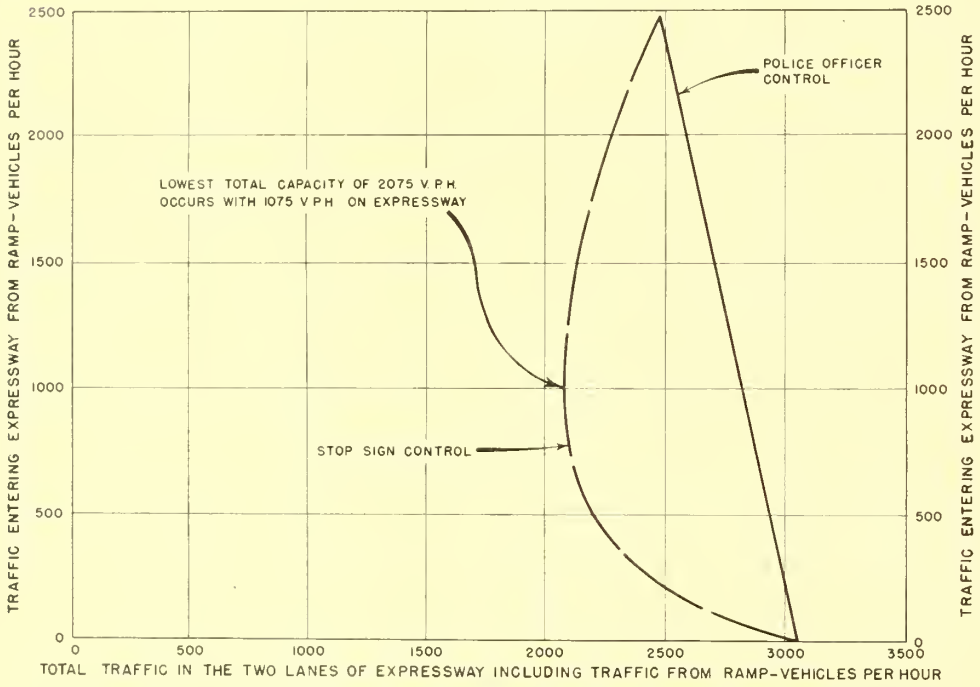

POLICE OFFICER CONTROL
LOWEST TOTAL CAPACITY OF 2075 V.P.H. OCCURS WITH 1075 V.P.H. ON EXPRESSWAY
STOP SIGN CONTROL
TRAFFIC ENTERING EXPRESSWAY FROM RAMP-VEHICLES PER HOUR
TOTAL TRAFFIC IN THE TWO LANES OF EXPRESSWAY INCLUDING TRAFFIC FROM RAMP-VEHICLES PER HOUR

Figure 48—Variation in ramp capacity with volume of traffic on an expressway: ramp 20 feet wide, with no acceleration area.

per hour, or only 90 percent of the volume needed to fill the expressway to its capacity.

Exit from expressway to ramp

In addition to the actual geometric features near the point where traffic leaves an expressway to enter a ramp, the maximum volume of traffic that can enter a ramp without causing unsatisfactory operating conditions on the expressway is influenced (1) by the total volume of traffic on the expressway, (2) by the percentage of this total volume that desires to use the ramp, and (3) by the percentage of commercial vehicles. Regardless of the general lay-out, within reasonable limits, 1,200 to 1,500 vehicles per hour can enter a ramp in one line if all of these vehicles and no other vehicles are in the lane adjacent to the ramp. Likewise, two lines of traffic or 2,400 per hour can enter a ramp if all of these vehicles are in the two lanes of the expressway adjacent to the ramp and the drivers in the second line can be sure that all vehicles in the first line are headed for the ramp.

A perfect segregation of the traffic bound for a ramp seldom occurs ahead of the ramp, especially when an expressway is loaded to capacity or near capacity volumes. Some through vehicles, especially those driven by the slow, cautious drivers or the drivers looking for destination signs, and the commercial vehicles, will be in the right-hand lane of the expressway. If a ramp is on the right, these vehicles occupy spaces in the lane that could otherwise be used by vehicles entering the ramp. They also cross the paths of vehicles entering the ramp from the second lane, thereby creating a hazardous condition whenever the traffic volume entering the ramp exceeds 1,200 to 1,500 vehicles per hour, which require the use of two lanes.

The tendency of some drivers of through vehicles to stay in the right-hand lane where other traffic leaves the expressway by a ramp on the right is illustrated by figure 49. The distribution by lanes at the low traffic volumes differs greatly from the distribution found on sections where there are no ramps (fig. 46). It also differs from the distribution at the approach to a ramp used by traffic entering an expressway (fig. 47). At the near-capacity volumes, however, the difference for the various conditions is much less marked, there being a tendency in all cases for traffic to be fairly evenly divided between the lanes.

CONCLUSIONS

Although the data presented in the foregoing discussion show ramp capacities for only a few specific conditions, application of the information relative to lane usage on expressways is of assistance in rationalizing the problems concerning ramp capacities under a wide variety of conditions.

The following conclusions briefly summarize the more important facts concerning ramp capacities:

1. The capacity of a ramp may be limited by either the width and alinement of the ramp, or by traffic and physical conditions at either terminal.

2. With the curvatures and traffic conditions that are generally present at ramp locations, it is usually difficult to obtain better than the equivalent of single-lane operation on the ramp. With such operation, the ramp capacity normally will not exceed 1,200 passenger cars per hour with a possible maximum of 1,500 passenger cars per hour when vehicles can keep moving at a speed of 15 to 20 miles per hour (minimum radius about 100 feet). Serious consideration should be given to the provision of additional ramps when the traffic demand exceeds 1,200 vehicles per hour on any one ramp, unless conditions required for safe two-lane operation can be satisfied. Conditions that must be present to enable a ramp to accommodate 1,200 vehicles per hour are:

   (a) Adequate ramp width for off-tracking and turning requirements of commercial vehicles (note: The 1,200 figure is for passenger cars and must be lowered to compensate the effect of commercial vehicles that are present. As a minimum, each commercial vehicle with dual tires has the effect of two passenger cars).

   (b) Adequate surface or shoulder width for disabled vehicles, especially on up ramps.

   (c) Adequate speed change areas, or excess capacity of the roadways connected by the ramp.

3. It is possible for a ramp and its terminals to handle traffic volumes exceeding 1,200 vehicles per hour under certain conditions. The following are some of the conditions that must be satisfied:

(a) Adequate surface width and radius of curvature (at least a 28-foot surface and a minimum radius of about 200 feet).

(b) At least one through lane on each highway entirely free of traffic other than that using the ramp, and a second lane on each highway sufficiently free of through traffic to accommodate the ramp volume which is in excess of 1,200 vehicles per hour (generally the exit and entrance to the ramp must have design characteristics similar to those at a Y intersection of two highways).

4. In considering the availability of spaces in the through traffic lanes into which vehicles from the ramp may enter, the following factors are important:

(a) At traffic volumes below 1,000 vehicles per lane per hour on a multilane highway, there is a marked tendency for traffic approaching an entering ramp to shift away from the lane adjoining the terminal of the ramp, thereby avoiding interference with vehicles from the ramp and providing a better opportunity for vehicles to enter the highway. When the volume in one direction on the highway exceeds 1,000 vehicles per hour per lane, however, this tendency to shift away from the lane adjoining the ramp does not occur, and traffic maintains approximately the same distribution between the lanes at the ramp terminal as on sections where there are no ramps. This is particularly true at locations where there are stop signs for vehicles entering from the ramp. Where there are acceleration areas without stop-sign control for vehicles from the ramp, a somewhat higher percentage of the through traffic will use the lane farthest from the ramp than normally uses this lane at locations removed from the influence of ramps.

(b) Because of the tendency of traffic to distribute itself in all lanes of a multilane highway during heavy traffic volumes, thus avoiding a more crowded condition in one lane than in another lane, many of the openings between vehicles in the lane or lanes that are not adjacent to the ramp terminal are shielded against occupancy by vehicles entering from the ramp. As a practical matter, it is impossible to fill all the available openings between cars on an expressway unless there are suitable acceleration areas where merging can be performed. Hence the volume of traffic that can enter from a ramp will seldom equal the amount by which the capacity of the expressway exceeds the volume of traffic on the expressway ahead of the ramp.

5. Where points of access and egress to and from the expressway are closely spaced, the resulting interchange of traffic between lanes may cause certain lengths of the expressway to fall within the category of weaving sections and in that way affect the capacity of the expressway and the ramps.

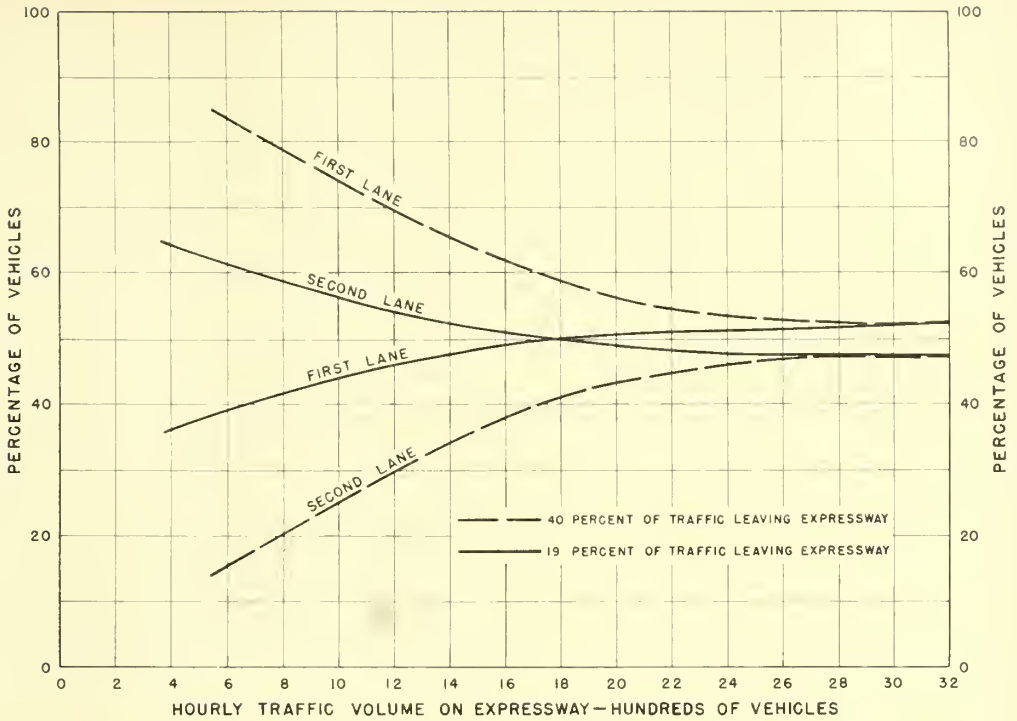


Figure 49.—Distribution of vehicles between traffic lanes on a four-lane expressway at approach to ramp where heavy volume leaves the expressway to the right (hourly traffic volume on expressway includes traffic leaving expressway at the ramp).

6. In selecting the hour of peak traffic movement, a knowledge of the distribution of traffic between the ramp and the expressway is of utmost importance. An hourly pattern for traffic using a ramp is of little worth without a similar pattern for the expressway or other through street.

7. For exit ramps, the number of vehicles that can enter the ramp is affected by the volume of through traffic using the right lane. Many slow-moving passenger cars and most commercial vehicles use this lane, thereby occupying spaces in the traffic stream that otherwise might be utilized by vehicles desiring to leave the expressway. For most installations, the volume of traffic using an exit ramp cannot exceed 1,200 passenger cars less the number of through vehicles occupying the lane adjacent to the ramp. Proper use of signs will aid greatly in minimizing the number of vehicles that must be deducted from the ramp capacity for this reason.

If an exit ramp is to accommodate more than 1,200 passenger cars an hour, the conditions enumerated under item 3 above must be satisfied.

Exit turn from an expressway at a grade separation. Two lines of traffic can enter a ramp if they are in the two lanes of the expressway adjacent to the ramp, and if the drivers in the second line can be sure that all vehicles in the first line are destined for the ramp.

Part VIII.—Relating Hourly Capacities to Annual Average Volumes and Peak Flows

Introduction

Throughout the foregoing discussion of highway capacities, traffic volumes have been expressed in terms of vehicles per hour. This time period was selected by the Committee because periods of congestion or heavy traffic movement usually are of relatively short duration, occupying only a small portion of the day. **An hour is the shortest period of time that conforms to current traffic counting practices although it is realized that, under certain conditions, counts for even shorter periods might be desirable.**

When trying to apply the hourly capacities to the solution of practical problems, the engineer often finds that complete and detailed hourly volume data are not available for the facility under consideration. Sometimes the only traffic information at hand is the annual average 24-hour volume, based either on a few counts scheduled throughout the year or estimated from counts made at nearby locations. In other instances, the only traffic volume information available may be a count for a single day, counts for a few scattered days during the year, or, as oftentimes occurs, counts for periods shorter than 24 hours. **Although the cost of obtaining comprehensive traffic data is relatively inexpensive, in comparison with the cost of any extensive improvement, it is oftentimes necessary to make use of very meager data in the solution of problems involving highway capacity. In such cases, a method for adjusting the available counts to obtain the needed hourly capacities becomes a matter of paramount importance, and a clear understanding of the variations which may be expected in the traffic load is necessary.**

Without this knowledge, the application of traffic-count data to the planning and design of new facilities or to the proper operation of existing facilities cannot be completely successful. It is the purpose of this chapter to supply information regarding these questions, which are so directly related to highway capacity and necessary for the proper use of the capacity data.

Relation of Annual Average Volume to Capacity

The most common measure used in reporting the traffic on a highway is the annual average daily traffic volume. Although such a figure is of value for many purposes, such as determining the type of surface that should be provided and in computing the total earnings derived from gasoline taxes by travel on the highway, it is not a direct measure of the number of needed traffic lanes or of other geometric features of the highway necessary to serve adequately the drivers using the highway.

Proper use of signs plays an important part in preventing or alleviating congestion.

Days during which the actual volume corresponds to the average day of the year occur much less frequently than is commonly supposed. A study of 48 selected rural highway locations in 45 States shows that the traffic volume representing the annual average daily volume is exceeded during 160 days of the 365 days in a year at the average location. At some locations, the average daily volume was exceeded on as few as 70 days a year and at other locations it was exceeded on as many as 228 days during a year.

It is therefore evident that facilities planned to accommodate only the traffic of an average day will be definitely overtaxed for a considerable portion of the year. In fact, at the average rural location the volume on certain days will be more than double the annual average. The practice of expressing capacities of various facilities in terms of average daily traffic without considering the variation in traffic flow and the relation between the average daily volumes and the peak hourly volumes has contributed in a large measure to the misunderstanding and lack of agreement regarding highway capacities.

Selection of Required Hourly Capacities From Continuous Counts

Closely related to the fluctuation in traffic flow during the various hours of the day, days of the week, and seasons of the year, is the selection of the specific hourly volume that should be used for design purposes or as the reasonable volume which an existing street or highway facility should be expected to accommodate. It is for these purposes that the time period of 1 hour assumes primary importance.

If a roadway facility is to be so designed that traffic will be properly served, consideration must be given to the brief but frequently repeated rush-hour periods. It is neither wise nor economical, however, to provide for the extreme hourly volumes of traffic that may occur but a few times during a year. The law of diminishing returns must be applied to fix the highest hourly volume which will justify the necessary expenditure of funds to provide the added capacity.

When hourly traffic counts for a full year are available for a highway under consideration, it is possible to show the yearly traffic pattern by arranging the hourly volumes in descending order of their magnitude. Such a table or graph greatly simplifies the problem of selecting the minimum hourly capacity which will adequately serve present traffic for the particular highway, or a future traffic demand as based on an assumed or estimated percentage increase.

An example of how yearly traffic patterns can be used to select the minimum hourly capacity which will adequately serve the traffic presently using the highway is presented in table 21. This table shows the yearly traffic patterns for two rural highway sections in the same State, both having an annual average volume of 3,000 vehicles per day. Data for Sunday and holiday traffic are included in the table because **there is no sound basis for any assumption that there is less need to provide adequate capacity for peak hours on Sundays and holidays than on weekdays.**

Although both roads have the same annual traffic volume, the peak hourly traffic volumes on road *B* are much higher than those on road *A* because of the greater seasonal and weekly fluctuation in traffic flow. On road *A*, 1 percent of the vehicles use the road when the hourly traffic volume is 500 vehicles or more. On road *B*, however, more than 20 percent of the vehicles, or about 200,000 per year, are traveling when the hourly traffic volume is

Table 21.—*Yearly traffic patterns for two rural highway sections on each of which the average annual daily traffic is 3,000 vehicles*

Hourly traffic volume	Percentage that hourly volume is of average 24-hour volume	Road *A*		Road *B*	
		Number of hours during year when hourly volume was exceeded	Percentage of total vehicles using road when hourly volume was exceeded	Number of hours during year when hourly volume was exceeded	Percentage of total vehicles using road when hourly volume was exceeded
3,000	100.0	0	0	0	0
1,300	43.3	0	0	0	0
1,200	40.0	0	0	6	0.69
1,100	36.7	0	0	15	1.64
1,000	33.3	0	0	29	2.99
900	30.0	0	0	62	5.88
800	26.6	0	0	100	8.50
700	23.3	1	0.06	153	12.43
600	20.0	5	.30	206	15.63
500	16.7	20	1.01	298	20.24
400	13.3	111	4.60	430	25.67
300	10.0	360	12.29	727	35.15
200	6.7	1,366	34.04	1,388	50.05
100	3.3	5,270	84.21	3,510	78.43
0	0	8,760	100.00	8,760	100.00

500 vehicles or more; and over 8 percent, or about 80,000 per year, are traveling when the hourly traffic volume is 800 or more vehicles.

Thus, if the practical capacity of road *A* is only 450 vehicles per hour it will be no more congested than will road *B* if the practical capacity of the latter road is 1,000 vehicles per hour. In both cases the practical capacity would be exceeded during about 30 hours of the year and about 3 percent of the total traffic using each road during the year would be inconvenienced by congestion during the 30 hours.

The yearly traffic patterns also show that relatively few drivers would be benefited by roadways of higher practical capacities than these, and an increasingly large percentage of drivers would be inconvenienced by roadways with only slightly lower practical capacities.

Ordinarily the foregoing procedure can be followed in the selection of a practical hourly capacity for an estimated future increase in traffic by merely changing the hourly traffic volumes shown in the first column of table 21 by the same percentage as the estimated increase in the annual volume. **Hourly volumes increase in about the same ratio as do average daily volumes on any roadway.**

Determining Required Hourly Capacity from Annual Volume

The preceding discussion and table 21 illustrate the wide difference that might exist in the peak hourly traffic flows on two highways having the same annual traffic volumes, and the resulting difference in hourly capacities of two highways necessary to provide approximately the same traffic service under the two conditions.

As an aid in relating the annual traffic volumes to the peak hourly flows, the results of an analysis of traffic count data for 171 stations in 48 States are summarized in table 22 and detailed in table 23. It is anticipated that these tables will be of value in estimating peak hourly flows on roads within the area of influence of these stations, or where similar conditions exist, when only the annual traffic volumes are known.

Figure 50 shows the average yearly traffic pattern for these 171 rural highway locations and an indication of the range in peak flows that occur at different locations. On a road

Table 22.—*Variations in traffic flow on main rural highways averaged for the various census regions (prewar data)*

Census region	Locations studied	24-hour volumes			Percentage of average 24-hour volume in certain hourly volumes during year				
		Average for year	Percentage of average in—		Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
			Maximum 24 hours	Tenth highest 24 hours					
		Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
New England	17	3,657	316.9	214.9	34.5	25.6	23.1	21.7	19.6
Middle Atlantic	10	13,884	241.4	190.2	21.9	17.2	16.0	15.2	14.2
South Atlantic:									
North portion	11	4,349	207.2	166.4	20.1	16.6	15.5	14.8	13.8
South portion	12	2,595	161.8	135.1	18.3	14.3	13.3	12.7	11.9
East North Central	22	2,864	263.7	200.0	29.0	19.6	17.8	16.6	15.3
West North Central	20	2,017	250.6	178.5	29.4	18.9	16.7	15.7	14.4
East South Central	12	2,420	187.2	151.9	23.7	17.6	16.3	15.6	14.3
West South Central	14	3,108	177.6	140.8	17.7	13.9	12.9	12.3	11.5
Mountain	36	1,940	216.1	165.9	23.7	16.0	14.6	13.9	12.8
Pacific	[1] 13	2,975	220.5	167.4	24.1	16.7	15.0	14.2	13.1
Total	167	3,370	228.9	172.6	24.9	17.8	16.2	15.3	14.1

[1] Does not include four stations in California where traffic is highly seasonal and not representative of most roads in the State.

Table 23.—*Variations in traffic flow on main rural highways*

Region and State	Recorder station No.	Route No.	Year starting	24-hour volumes			Percentage of average 24-hour volume in certain hourly volumes during year				
				Average for year	Percentage of average in—		Maximum hour	10th highest hour	20th highest hour	30th highest hour	50th highest hour
					Maximum 24 hours	10th highest 24 hours					
NEW ENGLAND:				*Vehicles*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Connecticut	6-WB	[1] 1	Mar. 31, 1939	6,813	287.7	244.3	29.7	25.6	24.0	23.4	22.0
	7-EB	1	do	6,811	297.2	221.4	26.7	21.7	19.6	18.5	17.5
	6 and 7	1	do	13,624	292.5	227.9	25.7	21.2	19.2	18.6	17.7
	17	5	do	8,313	155.3	136.8	13.3	12.0	11.5	11.2	10.8
Maine	3	27	Dec. 15, 1940	1,397	319.2	240.9	39.8	28.6	25.8	24.3	22.0
	2	1	Feb. 5, 1938	1,287	274.1	263.4	22.4	19.8	18.8	17.9	17.1
	2	1	Jan. 1, 1940	1,248	274.3	231.6	24.0	20.6	19.5	18.7	17.6
	4	17	do	436	354.8	183.9	39.4	21.6	19.5	18.1	16.7
Massachusetts	8	1	Apr. 30, 1938	7,363	288.2	246.9	24.6	22.2	20.7	19.9	18.2
	10	10	July 21, 1939	6,476	218.7	189.0	18.8	16.8	16.3	15.8	15.0
	1	8	Jan. 1, 1940	3,071	205.4	160.6	29.3	20.7	17.4	16.3	14.6
New Hampshire	2	3	May 12, 1940	6,635	263.3	198.8	28.5	20.6	18.7	17.8	16.4
	1	3	Jan. 1, 1940	1,410	517.2	277.8	55.2	43.0	37.5	32.8	27.5
	1	3	Sept. 18, 1937	1,360	647.6	279.9	60.4	48.4	38.7	34.1	28.9
	3	[2] 104	Oct. 22, 1939	535	298.1	227.1	34.7	25.4	23.7	22.8	20.9
Rhode Island	4	[2] 146	Jan. 1, 1940	4,609	197.2	166.9	26.3	21.3	19.6	18.5	16.6
	2	1	do	2,155	452.9	263.9	57.2	41.6	37.1	34.0	30.0
	2	1	June 4, 1938	1,931	466.8	238.0	68.3	43.8	37.9	34.2	31.4
	3	[2] 102	Jan. 1, 1940	330	240.9	215.2	62.7	27.6	26.1	24.8	22.7

See footnotes at end of table, p. 271.

TABLE 3.—Variations in traffic flow on main rural highways—Continued

Division and State	Station No.	Year starting	Average volumes: Average for year	Average volumes: Percentage of average in: Maximum 24 hours	Average volumes: Percentage of average in: 10th highest 24 hours	Percentage of average 24-hour volume in certain hourly volumes during year: Maximum hour	10th highest hour	20th highest hour	30th highest hour	50th highest hour
			Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
New England mountain:										
	A 1-2	2 Jan. 1, 1949	1,688	276.3	196.1	38.1	19.5	17.5	16.9	16.2
Vermont	A 1-2	2 Nov. 28, 1936	1,615	276.5	219.6	31.3	22.2	20.9	20.3	18.0
	C 14-2	[2] 14 Jan. 1, 1941	1,145	275.1	194.8	34.1	23.3	20.5	19.0	17.6
	B-6 1	7 Mar. 2, 1941	352	706.0	280.1	69.9	44.5	40.0	37.2	30.3
Middle Atlantic:										
	NB	25 Jan. 1, 1941	32,100	144.2	130.6	14.2	11.9	10.9	10.5	9.2
New Jersey (Bergen County	SB	25 do	30,150	159.9	137.8	13.3	11.4	11.1	10.9	10.3
	NB and SB	25 do	62,250	142.6	131.6	11.2	9.6	9.3	9.3	8.8
	NB	35 do	13,725	257.9	167.3	26.7	23.3	21.1	19.7	18.0
New Jersey (Edison Bridge)	SB	35 do	11,656	136.3	315.3	36.0	28.0	24.7	24.7	23.6
	NB and SB	35 do	25,381	284.3	231.6	24.0	19.1	17.3	16.2	15.4
	NB	35 do	11,286	229.3	181.4	30.1	20.7	18.4	15.9	8.9
New Jersey (Woodbridge)	SB	35 do	10,766	189.0	255.3	41.5	30.1	26.2	21.7	18.9
	NB and SB	35 do	22,052	209.2	211.5	26.8	18.0	17.0	15.9	13.6
	S 6	17 Feb. 25, 1940	7,594	291.7	255.1	21.1	19.6	18.8	18.3	17.6
New York	2-1	[2] 5 Jan. 1, 1940	4,794	156.4	138.7	15.4	11.7	11.3	11.1	10.5
	5-1	[2] 5 Dec. 31, 1938	4,458	329.2	211.5	27.2	22.7	19.9	18.6	17.4
	5-3	33 Jan. 1, 1939	2,843	186.1	165.5	20.7	17.9	16.9	16.0	14.4
	1	20 Jan. 1, 1941	6,112	291.3	212.7	23.9	20.0	17.2	16.2	15.3
	1	20 Nov. 20, 1937	4,395	311.1	209.6	28.1	19.3	17.2	16.0	14.9
Pennsylvania	17	[2] 53 Jan. 1, 1940	2,979	160.4	141.6	18.7	13.7	13.1	12.7	12.1
	4	6 July 24, 1937	1,231	261.9	203.7	27.8	20.1	20.1	18.3	17.1
South Atlantic (North portion):										
	C-NB	13 June 8, 1941	3,371	344.9	235.9	40.1	34.6	31.4	29.2	25.6
Delaware	C-SB	13 do	3,370	287.5	202.1	25.4	21.1	17.7	16.9	15.3
	C Comb	13 do	6,741	254.2	206.5	23.8	20.5	20.0	19.1	16.9
	E	13 May 1, 1941	3,430	201.3	158.4	17.7	14.4	12.9	12.1	11.3
	12	10 Jan. 22, 1938	7,250	227.0	171.1	22.3	18.2	16.6	15.5	14.3
Maryland	11	1 Jan. 1, 1940	5,157	215.5	171.6	27.3	20.4	18.8	17.6	16.2
	2	40 Apr. 3, 1937	6,030	225.5	190.3	23.5	16.8	16.1	15.4	14.7
	1	1 June 26, 1937	6,668	216.3	174.5	20.4	16.9	15.6	14.9	13.9
Virginia	2A	11 May 5, 1940	2,600	214.2	163.7	17.3	15.9	15.0	14.3	13.5
	4A	58 Jan. 31, 1939	2,470	151.2	142.7	13.6	12.6	11.9	11.6	11.0
	3	60 Dec. 8, 1939	7,285	170.5	131.0	12.5	11.2	10.6	10.5	10.2
West Virginia	1	[2] 10 June 4, 1939	1,727	169.2	141.9	17.6	15.7	14.9	14.4	13.7
	11	[2] 2 Jan. 1, 1940	1,258	234.7	178.3	25.5	20.2	17.6	17.1	15.7
South Atlantic (South portion):										
	4	90 Jan. 1, 1941	3,646	117.3	137.1	13.3	11.9	11.3	10.9	10.3
Florida	4	90 May 15, 1937	3,365	144.2	119.7	14.9	10.8	10.3	9.8	9.2
	3	41 Jan. 1, 1938	1,668	147.3	129.0	13.8	11.6	11.2	10.8	10.4
	1	90 Nov. 27, 1937	749	144.9	125.6	24.4	13.0	12.3	11.2	10.3
	3	29 Jan. 1, 1940	4,307	153.5	140.4	16.7	15.6	14.7	14.1	13.3
Georgia	1	41 Jan. 1, 1939	3,238	142.9	126.7	14.2	12.7	12.2	11.5	11.1
	12	84 do	632	170.6	135.9	20.9	15.0	13.8	12.2	11.4
	3	29 do	4,296	174.4	135.5	25.0	16.8	14.5	13.9	12.6
North Carolina	4	19 Feb. 25, 1939	2,540	163.2	144.2	16.4	15.2	13.9	13.2	12.4
	12	158 Jan. 1, 1941	1,414	218.8	157.6	20.6	17.1	15.9	15.4	14.6
South Carolina	5	29 Feb. 20, 1937	3,936	154.5	133.4	15.9	15.0	14.5	13.9	13.0
	7	13 Jan. 1, 1940	3,268	174.7	137.2	20.4	14.7	14.0	13.4	12.9
	2	15 Dec. 4, 1937	1,583	151.3	129.6	16.9	13.4	12.4	11.9	11.2
East North Central:										
	1	45 Sept. 27, 1936	1,057	413.7	369.7	31.8	29.6	28.1	27.1	25.9
Illinois	2	66 Jan. 24, 1937	3,937	233.2	185.3	25.0	18.3	16.1	15.1	13.8
	7	50 Dec. 18, 1937	3,210	171.9	126.4	15.9	14.1	13.1	12.5	11.3
	2A	20 Aug. 28, 1937	3,490	296.8	186.5	25.8	18.3	17.2	16.1	13.7
	34A	30 Dec. 21, 1940	3,198	294.2	186.2	17.6	15.6	14.2	13.4	12.3
Indiana	59A	40 Jan. 15, 1938	3,125	233.2	161.1	33.2	14.9	13.9	12.8	12.4
	42A	52 July 3, 1937	3,071	291.7	179.0	43.2	17.3	15.5	14.4	13.8
	72A	31 Jan. 15, 1938	2,293	213.8	165.9	20.4	15.3	13.4	13.0	12.1
	676	27 Oct. 2, 1937	3,151	235.3	200.6	53.9	20.7	17.2	16.3	15.3
Michigan	678	23 Jan. 1, 1941	1,538	336.0	246.8	36.4	28.3	26.0	24.3	23.0
	678	23 Jan. 1, 1939	1,200	300.9	267.5	44.8	31.2	26.2	24.8	22.5
	27	25 Feb. 18, 1939	3,928	213.7	156.9	23.1	15.3	13.8	13.3	12.5
Ohio	25	42 Apr. 12, 1939	3,645	214.4	158.5	21.2	14.7	13.6	13.1	12.4
	28	75 June 25, 1939	1,055	180.6	158.2	19.9	16.8	15.4	14.3	13.4
	2 and 3	[2] 41 Jan. 8, 1938	5,674	305.9	220.2	29.1	22.2	18.9	18.1	16.3
	2-WB	[2] 41 do	2,817	405.6	241.8	43.7	30.7	25.6	22.9	20.2
	3-EB	[2] 41 do	2,857	281.6	220.3	22.1	19.7	17.8	16.4	15.3
	3 EB	[2] 41 Jan. 1, 1941	3,921	439.8	217.0	30.4	23.4	20.5	18.9	16.0
	13	[2] 41 Oct. 4, 1939	4,870	263.6	234.8	26.0	22.4	21.1	19.9	18.1
Wisconsin	14	10 Jan. 1, 1940	2,953	248.4	211.1	30.0	18.5	17.2	15.8	14.6
	18	[2] 13 do	2,780	317.5	229.4	32.1	21.3	19.6	18.0	16.8
	5	[2] 41 do	2,199	203.6	163.8	31.4	17.1	15.1	13.2	12.4
	15	45 Jan. 6, 1939	2,037	289.8	211.1	24.5	20.1	18.4	16.7	15.5
	10	10 Jan. 9, 1937	1,632	364.2	242.3	28.5	19.9	19.4	18.0	17.0
	16	[2] 13 Jan. 1, 1940	1,044	294.1	189.1	24.3	19.6	18.0	16.3	14.4
	20	[1] P&S do	282	312.4	206.7	40.8	29.4	26.2	23.4	20.2
West North Central:										
Iowa	601	65 Jan. 1, 1938	3,539	223.3	157.4	26.7	16.6	14.9	14.2	13.2
	601	65 Dec. 19, 1936	3,290	232.4	154.2	18.1	14.9	14.3	13.4	12.3
	9-616	69 Jan. 1, 1941	1,417	243.2	184.3	25.5	16.8	15.7	14.9	13.8
Kansas	5	24 Aug. 14, 1938	2,183	185.3	155.0	14.9	13.1	12.6	12.4	11.9
	3	508 Feb. 18, 1939	2,059	164.9	146.2	17.1	14.3	13.3	12.2	11.5
	6	24 Jan. 1, 1940	991	162.9	147.7	18.7	12.9	12.0	11.7	11.2
Minnesota	157	212 Mar. 20, 1937	4,875	221.5	191.1	22.9	19.7	18.7	17.8	16.6
	159	10 Sept. 11, 1937	3,730	393.9	226.2	33.7	26.1	22.9	19.7	17.4
	175	52 July 3, 1937	872	264.8	183.0	28.9	22.2	18.2	17.4	15.0
Missouri	9	66 Jan. 23, 1939	5,220	339.3	257.4	28.4	25.1	22.1	21.4	20.3
	12	69 Jan. 1, 1939	3,307	200.2	165.4	28.9	16.1	14.8	14.3	13.7
	5	54 July 17, 1937	1,708	433.4	255.1	46.3	31.6	28.3	27.1	23.7
Nebraska	5	6 Jan. 8, 1938	2,128	238.9	166.8	48.8	28.0	17.0	14.9	13.9
	A4	77 June 15, 1940	2,096	326.9	139.1	27.4	14.6	11.4	9.9	9.2
	2	30 Jan. 8, 1938	1,619	200.9	178.9	16.9	13.8	13.3	12.9	12.6
North Dakota	111	10 Jan. 1, 1940	585	354.2	194.7	30.1	20.3	17.8	16.9	15.8
	102	[2] 1 Feb. 1, 1939	356	241.2	148.6	44.4	19.1	17.4	16.0	14.0
	103	2 Oct. 18, 1937	352	237.2	173.6	28.4	18.8	18.8	17.3	14.2
South Dakota	104A	77 Jan. 1, 1940	1,966	197.7	160.9	21.8	16.5	14.8	14.2	13.2
	101	14 May 15, 1937	982	290.8	199.1	54.4	18.2	16.8	16.3	15.7
	106	18 Dec. 31, 1938	479	186.0	140.3	27.6	14.4	13.4	12.9	11.9

See footnotes at end of table, p. 271.

Table 23.—*Variations in traffic flow on main rural highways*—Continued

Region and State	Recorder station No.	Route No.	Year starting	24-hour volumes			Percentage of average 24-hour volume in certain hourly volumes during year				
				Average for year	Percentage of average in		Maximum hour	10th highest hour	20th highest hour	30th highest hour	50th highest hour
					Maximum 24 hours	10th highest 24 hours					
EAST SOUTH CENTRAL:				*Vehicles*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Alabama	5	11	Jan. 1, 1940	5,718	159.0	147.8	18.9	16.7	16.1	15.4	14.3
	4	78	Dec. 25, 1937	1,073	203.1	155.8	27.9	19.6	18.6	17.9	16.7
	2	72	Jan. 1, 1939	531	154.4	134.8	27.7	17.5	15.4	14.9	13.7
Kentucky	7	31W	Jan. 1, 1941	7,937	166.1	139.7	19.6	15.9	15.1	14.6	13.6
	9	41	Dec. 29, 1940	2,086	186.5	143.3	23.8	13.6	12.4	12.2	10.6
	12	23	May 4, 1941	1,323	166.7	141.5	16.3	14.6	13.5	12.6	11.7
Mississippi	3	51	Jan. 1, 1940	1,871	114.6	124.9	17.0	13.3	12.5	12.0	11.5
	8	45	do	1,671	177.4	128.1	22.9	16.6	15.1	14.0	12.7
	10	11	do	1,617	165.1	156.0	29.6	17.2	16.3	15.0	13.8
Tennessee	1	31W	Apr. 21, 1939	3,425	142.1	135.5	17.8	14.7	13.7	13.1	12.5
	3	[2] 71	July 14, 1939	990	410.4	274.3	44.1	36.5	31.9	30.6	27.8
	4	57	Feb. 9, 1941	794	170.8	141.3	19.1	15.4	14.9	14.4	13.0
WEST SOUTH CENTRAL:											
Arkansas	14	61	July 23, 1940	3,057	159.7	141.6	12.6	11.0	10.7	10.5	10.0
	13	70	Jan. 1, 1939	2,321	148.3	133.6	13.7	11.4	11.1	10.7	10.3
	17	[2] 22	Jan. 28, 1940	1,501	143.4	128.1	15.4	12.3	12.1	11.7	10.7
Louisiana	11	63	Jan. 1, 1939	311	305.5	172.7	31.8	23.5	19.9	18.0	15.4
	4	90	Apr. 24, 1937	4,226	143.1	125.6	15.4	11.7	11.1	10.6	10.0
	1	79	Dec. 25, 1937	3,304	158.6	124.5	16.0	13.4	12.0	11.6	11.0
	14	90	Jan. 1, 1940	3,159	139.6	127.5	14.8	12.0	11.4	11.0	10.6
Oklahoma	6	66	Jan. 1, 1939	4,291	170.9	126.9	16.0	12.3	11.9	11.2	10.5
	5	77	Feb. 27, 1937	2,259	259.0	180.3	19.0	16.7	15.7	15.3	14.0
	1	66	May 15, 1937	2,111	196.0	158.1	16.6	14.8	13.0	12.5	11.8
Texas	1	80	July 7, 1939	9,053	154.1	133.3	17.2	13.4	12.4	12.0	11.5
	4	77	Jan. 1, 1941	5,180	181.2	145.9	19.4	14.6	13.3	12.9	12.0
	4	77	Jan. 1, 1938	4,049	151.9	143.5	16.6	14.2	13.3	12.8	12.0
	5	80	Dec. 19, 1936	2,427	163.6	140.0	16.6	13.9	12.9	12.3	11.5
	8	81	Mar. 20, 1937	875	177.9	135.0	24.3	14.3	12.9	12.3	11.3
MOUNTAIN:											
Arizona	1	80	Jan. 1, 1941	8,757	157.4	130.1	16.3	12.2	11.7	11.2	10.5
	1	80	July 7, 1939	7,174	172.1	131.2	22.9	13.0	11.9	11.5	10.9
	4	60	Jan. 28, 1939	1,743	177.1	140.4	15.8	12.4	11.2	10.7	10.0
	2	80	Jan. 1, 1940	1,036	177.9	133.5	14.1	11.9	11.3	10.9	10.4
Colorado	11	85	June 26, 1938	5,472	138.3	129.7	14.1	12.4	11.7	11.1	10.5
	3	85	Feb. 27, 1937	4,334	214.6	173.7	21.9	16.0	15.0	14.3	13.4
	14	160	Jan. 1, 1940	1,286	327.9	151.2	46.0	18.4	14.7	14.5	12.4
	14	160	Jan. 1, 1941	1,224	319.9	160.0	46.1	17.4	15.9	13.6	12.4
	7	34	do	731	203.8	173.2	21.6	15.3	14.6	14.2	13.1
Idaho	2	30	Apr. 3, 1937	3,085	179.2	149.5	15.8	13.9	13.1	12.8	12.1
	1	10	Jan. 1, 1938	2,438	460.8	222.8	36.6	28.9	23.4	21.0	19.4
	3	30	do	2,290	165.9	139.0	17.0	12.2	11.5	11.3	10.6
Montana	A11	10	Dec. 3, 1939	2,777	213.4	165.0	18.4	13.7	12.9	12.4	11.6
	4	10	Oct. 29, 1938	982	232.8	177.1	19.9	17.4	15.0	14.5	13.5
	A7	91	June 30, 1939	495	280.2	196.2	36.0	22.2	18.8	18.0	16.2
Nevada	110	93	Jan. 1, 1940	1,752	239.1	139.3	19.1	16.2	13.8	11.4	10.7
	101	50	Nov. 6, 1937	1,469	165.2	130.6	20.5	15.7	12.8	12.3	11.0
	107	40	June 5, 1937	755	207.9	180.8	18.3	14.4	13.6	13.1	12.2
New Mexico	2	85	Jan. 1, 1941	3,375	173.7	146.2	15.2	13.7	13.2	12.8	12.4
	10	285	do	2,185	167.0	130.9	15.1	11.4	11.2	10.6	10.3
	8	[2] 28	do	1,804	165.2	140.2	18.6	12.5	11.9	11.6	11.3
	6	66	Jan. 15, 1938	1,574	194.5	160.0	18.9	15.1	14.6	13.7	12.7
	7	70	Aug. 7, 1937	1,461	179.6	154.8	20.7	11.8	11.4	11.1	10.3
	3	64	Jan. 1, 1941	1,456	191.7	167.6	17.7	13.7	12.7	12.5	11.9
	4	54	do	1,391	207.7	166.1	22.4	13.9	13.2	12.9	11.9
	11	70	do	1,324	204.7	152.5	19.5	13.6	12.8	12.3	11.6
	1	85	June 12, 1937	1,216	238.3	184.8	23.0	17.2	16.0	14.9	13.7
	1	85	Jan. 1, 1941	1,059	192.8	168.1	28.0	14.8	13.9	13.2	12.7
	5	[2] 18	do	1,163	148.8	126.8	23.9	13.7	12.9	12.4	11.7
	9	54	do	946	239.6	187.9	28.9	19.2	18.0	16.6	15.8
	9	54	Jan. 8, 1938	751	241.5	165.6	30.5	20.4	17.0	16.4	14.8
Utah	302	50	Jan. 1, 1941	4,422	231.5	164.4	17.7	15.4	14.5	14.2	13.4
	302	50	July 10, 1937	2,443	230.9	172.4	20.9	16.9	15.1	14.4	13.7
	301	40	Jan. 1, 1941	2,111	355.0	247.7	29.2	26.4	23.1	21.9	19.6
	301	40	Nov. 13, 1937	1,766	301.1	260.0	41.3	29.0	26.2	25.4	22.8
	305	89	Jan. 1, 1941	833	208.8	164.3	31.0	15.8	14.2	13.6	12.5
	308	40	do	557	234.3	210.4	27.8	19.7	18.9	17.7	16.7
Wyoming	204	30	do	1,647	233.3	198.7	31.8	17.7	16.6	15.8	14.3
	204	30	Jan. 1, 1939	1,257	234.4	197.9	26.6	18.8	17.1	16.5	15.0
	203	87	Jan. 1, 1940	1,551	180.3	161.3	29.8	15.4	15.0	14.5	13.0
	205	20	Jan. 1, 1941	1,375	216.7	152.7	20.1	13.3	11.8	11.0	9.5
	205	20	May 19, 1939	1,309	230.2	149.7	54.7	15.0	13.7	13.1	12.3
	206	30	Jan. 1, 1941	1,365	220.7	173.4	19.8	16.7	15.1	13.8	12.1
	207	14–16	do	555	286.5	253.5	30.8	24.7	22.1	20.2	16.0
PACIFIC:											
California	1	99	do	7,692	207.0	149.3	17.1	13.7	12.8	12.2	11.0
	1	99	July 10, 1937	5,815	197.2	159.9	18.9	15.3	13.5	12.4	10.8
	9 [4]	[2] 43	Jan. 1, 1941	5,228	254.0	236.0	27.6	24.7	23.1	22.4	
	2	99	do	3,532	211.4	164.8	17.8	14.3	12.9	12.1	
	2	99	Feb. 20, 1937	2,281	173.7	147.5	12.8	11.4	10.6	10.3	9.9
	6 [4]	[2] 43	Jan. 1, 1941	3,434	435.0	333.4	44.2	39.2	36.5	32.3	
	10	60	do	3,285	171.5	152.5	16.3	14.4	13.3	12.2	
	5	101	do	2,004	309.8	193.4	26.3	19.9	17.4	16.6	
	3	[2] 50	do	1,570	161.1	136.9	15.7	11.9	11.2	10.8	
	3	[2] 50	Aug. 6, 1939	1,444	183.7	139.9	54.4	12.5	11.4	11.1	10.6
	7	[2] 153	Jan. 1, 1941	955	159.0	128.5	30.3	14.8	13.1	12.7	
	8 [4]	[3] 201	do	814	291.1	224.8	49.4	33.4	29.0	26.5	
	4 [4]	[2] 42	do	711	606.0	485.5	80.0	59.5	53.9	50.6	
Oregon	5	99E	do	4,729	255.8	172.9	21.9	17.3	15.2	14.5	13.7
	5	99E	Jan. 1, 1940	4,054	285.2	167.9	26.4	15.7	14.6	13.9	12.8
	3	30	Nov. 27, 1937	1,252	248.5	175.2	29.2	17.6	16.6	15.6	13.7
	2	99	Jan. 1, 1940	915	237.8	157.0	24.4	18.4	16.3	14.9	13.9
Washington	4	99	Jan. 1, 1941	5,520	205.7	160.5	19.9	16.7	14.3	13.4	12.6
	4	99	Dec. 11, 1937	3,479	224.8	171.3	23.7	19.3	16.8	15.6	14.2
	1	99	Dec. 28, 1937	3,590	255.8	207.9	28.4	20.9	19.6	18.5	16.9
	3	101	Sept. 11, 1937	3,385	269.5	207.3	29.3	23.0	19.0	17.8	16.5
	10	10	Apr. 10, 1937	3,233	182.0	168.5	15.3	13.6	13.1	12.6	12.1

[1] Merritt Parkway.
[2] State routes. Others are U S numbered routes.
[3] County trunk highway.
[4] Traffic highly seasonal and not representative of most roads in the State. Not included in regional averages.

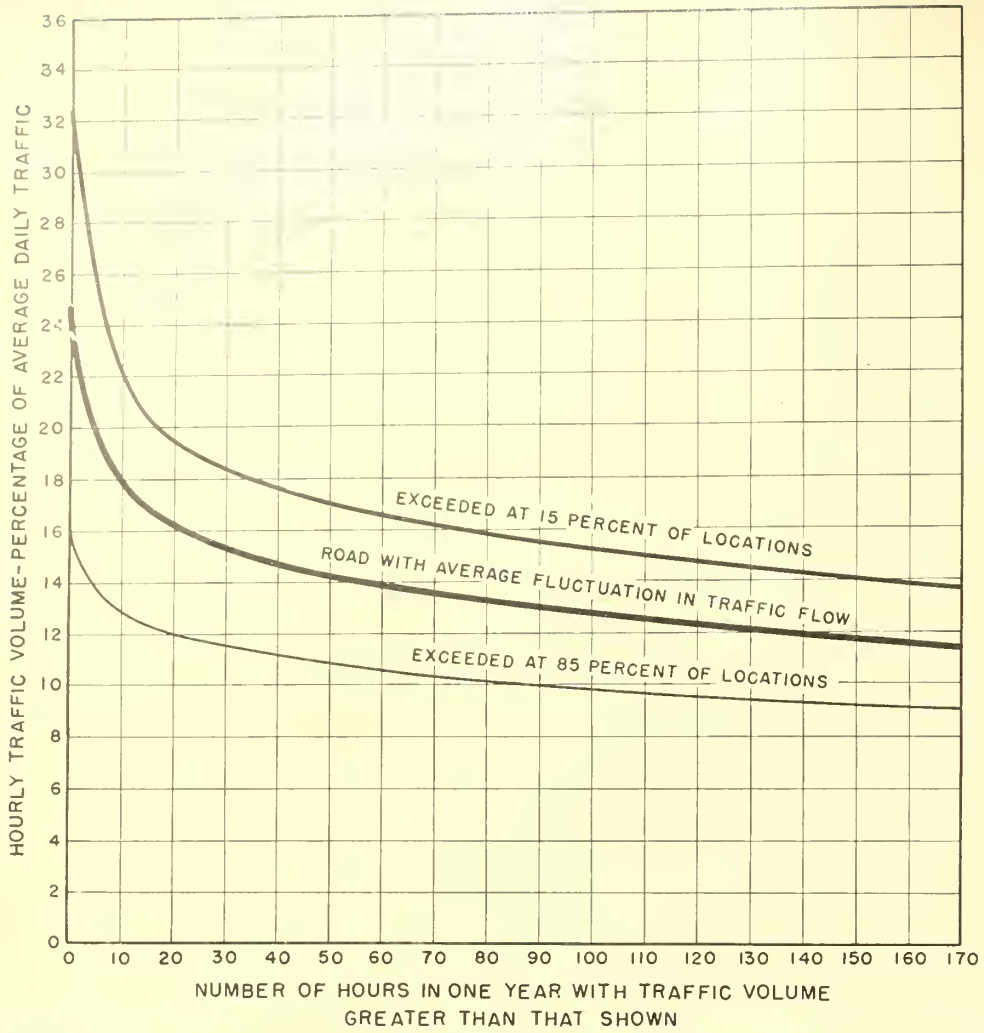

HOURLY TRAFFIC VOLUME-PERCENTAGE OF AVERAGE DAILY TRAFFIC
EXCEEDED AT 15 PERCENT OF LOCATIONS
ROAD WITH AVERAGE FLUCTUATION IN TRAFFIC FLOW
EXCEEDED AT 85 PERCENT OF LOCATIONS
NUMBER OF HOURS IN ONE YEAR WITH TRAFFIC VOLUME GREATER THAN THAT SHOWN

Figure 50.—Relation between peak hourly flows and annual average daily traffic on rural highways.

with the average fluctuation in traffic flow, the peak hourly volume is about 25 percent of the annual average daily volume. At 15 percent of the locations, however, the peak hourly volume during 1 year exceeded 32 percent of the average daily volume, whereas at another 15 percent of the locations the peak hourly volume was less than 16 percent of the average daily volume. At all locations, there are relatively few hours during the year when the traffic volume greatly exceeds the volume that occurs frequently or for a large number of hours each year. This would be more evident were the curves in figure 50 extended to include the 8,760 hours of a full year, in which case they would reach or approach the zero base line at a distance to the right equivalent to about 51 times the width of the chart.

Thirtieth Highest Hour a Practical Criterion of Needed Capacity

The relation between the peak hourly flows and the annual average daily traffic on rural highways, as shown by figure 50, serves as a guide when selecting a reasonable hourly volume for the design of a highway. Looking first at the curve showing the traffic fluctuation on the average rural road, it will be seen that the peak hour of the year will require a roadway having over twice the capacity that would be necessary if some degree of congestion could be tolerated during 130 hours of the year. To illustrate, consider a proposed highway for which an annual average daily volume of 4,200 vehicles is anticipated. To provide for the highest hourly volume during the year would require a capacity equal to 25 percent of 4,200, or 1,050 vehicles per hour. A facility capable of accommodating the traffic during all except the 130 highest hours of the year would require a capacity of only about 500 vehicles per hour.

During the highest 130 hours of the year, however, a very large number of drivers would be inconvenienced by congestion if the practical capacity were only 500 vehicles per hour. By planning for the volume during the 120th highest hour instead of the 130th, the change in the required design would be inconsequential because the difference in the two hourly volumes is negligible. Continuing with this procedure, **it will be found that at about the thirtieth highest hour, the slope of the curve changes rapidly, and it is at this point that the ratio of benefit to expenditure is near the maximum.**

Providing for an hourly traffic volume that is not exceeded at least 30 times a year will show an extremely small return in terms of driver benefit, whereas little will usually be saved in the construction cost and a great deal lost in driver benefit if provision is not made for the fiftieth highest hourly traffic volume of the year. It is for this reason that a design which will accommodate the fiftieth highest hourly traffic volume of the year can usually be justified, whereas a design to accommodate a traffic volume greater than that occurring during the thirtieth highest hour is generally not warranted.

Since this analysis is based on the average fluctuation in traffic volume for many highways, the results are not necessarily applicable to every location. However, by following a similar approach for the locations with high and low fluctuations in traffic flow, as represented by the upper and lower curves of figure 50, it can be shown that for these cases, and hence for most cases, the thirtieth highest hourly volume for the year is generally a reliable criterion of the needed capacity for which it is most practical to design. This criterion was recently adopted by the American Association of State Highway Officials as a design policy for the National System of Interstate Highways.

The strict adherence to the application of this criterion, however, will not always result in the best engineering practice. There are numerous specific locations where a design to accommodate some hourly volume other than the thirtieth highest of the year would be proper. For this reason, and because adequate traffic counts on which to base the design of a project are relatively inexpensive compared with the cost of construction, **the results of a continuous hourly traffic count for a full year should be available for locations at or near each place where a major construction or reconstruction project is contemplated.**

By considering the shape of the yearly traffic pattern—the curve found by arranging all the hourly volumes of one year in descending order of their magnitude—the most feasible hourly volume that should be used for design purposes can be determined.

In connection with the selection of the design volume, future increases in the traffic volume due to the normal increase in travel and induced traffic due to the improvement of a facility must be considered, together with the estimated life of the contemplated construction and the traffic capacities of the various possible designs. If, for example, the thirtieth highest hourly volume in 20 years would require a four-lane highway, whereas the fiftieth highest volume would only require a two-lane highway, it would be most practical to provide the two-lane highway using the fiftieth highest hourly volume for design purposes. If, however, a four-lane highway would be needed within 10 years, it may still be most feasible to construct only a two-lane road at the present time and make the necessary provision so that the two additional lanes can be constructed for a reasonable cost at some future date. In this case, the alinement

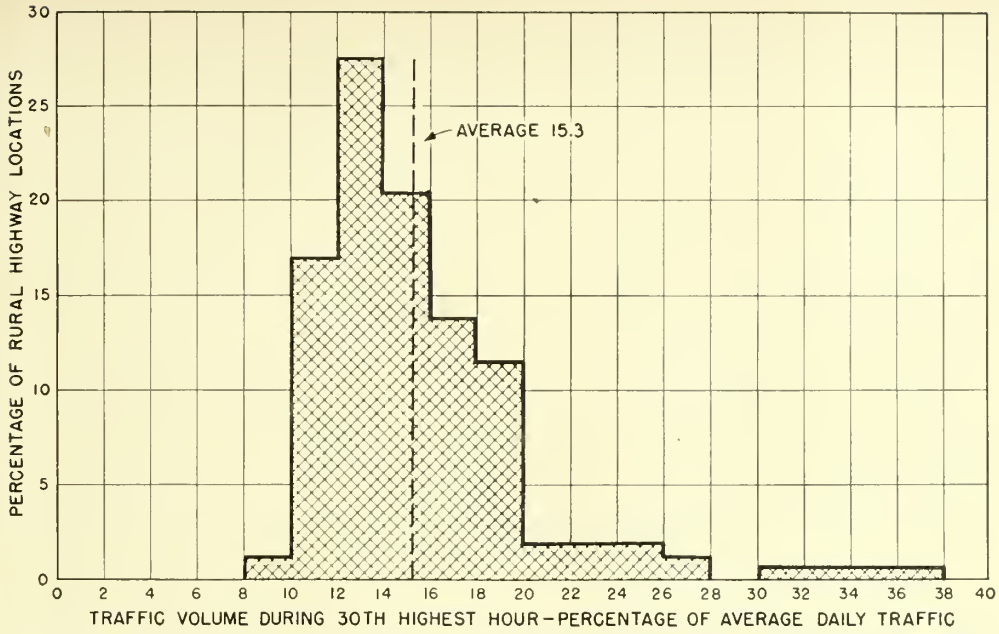

PERCENTAGE OF RURAL HIGHWAY LOCATIONS
AVERAGE 15.3
TRAFFIC VOLUME DURING 30TH HIGHEST HOUR—PERCENTAGE OF AVERAGE DAILY TRAFFIC

Figure 51.—Distribution of locations by percentage of average daily traffic during thirtieth highest hour of the year.

used might be such that the two-lane road would adequately serve traffic for a period of 10 years if in relatively flat terrain, whereas, if the road was located in rough terrain it might be more economical to build the two-lane road to the same alinement as a four-lane road (spending no funds to provide passing sight distances) even though the four-lane construction might then be required in less than 10 years.

Relation of Thirtieth Highest Hour to Annual Average Daily Traffic

The range in the relation between the thirtieth highest hourly volume and the annual average daily volume at rural locations is shown by figure 51. At nearly half of the locations, the thirtieth highest hour was between 12 and 16 percent of the annual 24-hour average. In view of the emphasis which has previously been placed on 10 percent as a peak-hour percentage, it is interesting to note that the thirtieth highest hour was under 10 percent at less than 2 percent of the locations and over 10 percent at 98 percent of the locations. Although it is true that on any single day the maximum hourly volume is likely to be close to 10 percent of the traffic on that day, **the use of a 10-percent value for design purposes is sound only where there is an exceptionally low seasonal and day-to-day variation in traffic flow.**

Only at relatively few locations is the thirtieth highest hourly volume an exceptionally high percentage of the annual average daily volume, as shown in figure 51 by the areas on the right which are above 20 percent. It might appear that these percentages are too high for use as design criteria and that they are probably too exceptional to be repeated year by year at the same locations. This is not the case, however, because the results of a study of the yearly patterns for different years at a number of locations (table 24) show that **the thirtieth highest hourly volume on a percentage basis changes very little from year to year.**

The consistency of the thirtieth highest hour as a part of the yearly traffic pattern is shown by figure 52. Data for two different years at 24 rural highway locations were summarized, with the thirtieth highest hour expressed as a percentage of the annual average 24-hour traffic volume. From this illustration it is **immediately evident that the many factors which operate to influence traffic at a particular location do not materially change the thirtieth highest hour percentage.** For example, locations 1 and 2 maintain their unusually high percentages despite the passage of 2 and 3 years, respectively. At location number 10 with a 39-percent increase in annual traffic over a 4-year period, the traffic volume in the thirtieth highest hour, expressed as a percentage of average daily traffic, changed from 16.0 to 16.2 percent. The comparison is just as favorable for lightly as for heavily traveled roads, as will be seen by noting the relative heights of the two bars for location 8 and those for location 20. In a few cases, average daily traffic volumes have dropped, as at locations 6, 12, and 16, but this has not resulted in any very marked change in the percentages representative of the thirtieth highest hours. For all 23 locations combined, there is an average difference of 1.1 percent between the

Table 24.—*Comparison of variations in traffic flow at the same location for different years*

State	Recorder station No.	Route No.	Year starting	24-hour volumes			Percentage of average 24-hour volume in certain hourly volumes during year				
				Average for year	Percentage of average in—		Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
					Maximum 24 hours	Tenth highest 24 hours					
				Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Rhode Island	2	1	June 4, 1938	1,931	466.8	238.0	68.3	43.8	37.9	34.2	31.4
			Jan. 1, 1940	2,155	452.9	263.9	57.2	41.6	37.1	34.0	30.0
New Hampshire	1	3	Sept. 18, 1937	1,360	647.6	279.9	60.4	48.4	38.7	34.1	28.9
			Jan. 1, 1940	1,410	517.2	277.8	55.2	43.0	37.5	32.8	27.5
Michigan	678	23	Jan. 1, 1939	1,200	300.9	267.5	44.8	31.2	26.2	24.8	22.5
			Jan. 1, 1941	1,538	336.0	246.8	36.4	28.3	26.0	24.3	23.0
Utah	301	40	Nov. 13, 1937	1,766	301.1	260.0	41.3	29.0	26.2	25.4	22.8
			Jan. 1, 1941	2,111	355.0	247.7	29.2	26.4	23.1	21.9	19.6
Vermont	A-12-2	2	Nov. 28, 1936	1,615	276.5	219.6	31.3	22.2	20.9	20.3	18.0
			Jan. 1, 1940	1,688	236.3	196.1	38.1	19.5	17.5	16.9	16.2
Maine	2	1	Feb. 5, 1938	1,287	274.1	263.4	22.4	19.8	18.8	17.9	17.1
			Jan. 1, 1940	1,248	274.3	231.6	24.0	20.6	19.5	18.7	17.6
Wisconsin	3-E B	41 [1]	Jan. 8, 1938	2,857	281.6	220.3	22.1	19.7	17.8	16.4	15.3
			Jan. 1, 1941	3,921	439.8	217.0	30.4	23.4	20.5	18.9	16.0
New Mexico	9	54	Jan. 8, 1938	751	241.5	165.6	30.5	20.4	17.0	16.4	14.8
			Jan. 1, 1941	946	239.6	187.9	28.9	19.2	18.0	16.6	15.8
Wyoming	204	30	Jan. 1, 1939	1,257	234.4	197.9	26.6	18.8	17.1	16.5	15.0
			Jan. 1, 1941	1,647	233.3	198.7	31.8	17.7	16.6	15.8	14.3
Pennsylvania	1	20	Nov. 20, 1937	4,395	311.1	209.6	28.1	19.3	17.2	16.0	14.9
			Jan. 1, 1941	6,112	291.3	212.7	23.9	20.0	17.2	16.2	15.3
Washington	4	99	Dec. 11, 1937	3,479	224.8	171.3	23.7	19.3	16.8	15.6	14.2
			Jan. 1, 1941	5,520	205.7	160.5	19.9	16.7	14.3	13.4	12.6
Utah	302	50	July 10, 1937	3,443	230.9	172.4	20.9	16.9	15.1	14.4	13.7
			Jan. 1, 1941	4,422	231.5	164.4	17.7	15.4	14.5	14.2	13.4
Oregon	5	99E	Jan. 1, 1940	4,054	285.2	167.9	26.4	15.7	14.6	13.9	12.8
			Jan. 1, 1941	4,729	255.8	172.9	21.9	17.3	15.2	14.5	13.7
Colorado	14	160	Jan. 1, 1940	1,286	327.9	151.2	46.0	18.4	14.7	14.5	12.4
			Jan. 1, 1941	1,224	319.9	160.0	46.1	17.4	15.9	13.6	12.4
Iowa	601	65	Dec. 19, 1936	3,290	232.4	154.2	18.1	14.9	14.3	13.4	12.3
			Jan. 1, 1938	3,539	223.3	157.4	26.7	16.6	14.9	14.2	13.2
New Mexico	1	85	June 12, 1937	1,216	238.3	184.8	23.0	17.2	16.0	14.9	13.7
			Jan. 1, 1941	1,059	192.8	168.1	28.0	14.8	13.9	13.2	12.7
Texas	4	77	Jan. 1, 1938	4,049	151.9	143.5	16.6	14.2	13.3	12.8	12.0
			Jan. 1, 1941	5,180	181.2	145.9	19.4	14.6	13.3	12.9	12.0
California	1	99	July 10, 1937	5,815	197.2	159.9	18.9	15.3	13.5	12.4	10.8
			Jan. 1, 1941	7,692	207.0	149.3	17.1	13.7	12.8	12.2	11.0
Wyoming	205	20	May 19, 1939	1,309	230.2	149.7	54.7	15.0	13.7	13.1	12.3
			Jan. 1, 1941	1,375	216.7	152.7	20.1	13.3	11.8	11.0	9.5
Arizona	1	80	July 7, 1939	7,174	172.1	131.2	22.9	13.0	11.9	11.5	10.9
			Jan. 1, 1941	8,757	157.4	130.1	16.3	12.2	11.7	11.2	10.5
California	2	99	Feb. 20, 1937	2,281	173.7	147.5	12.8	11.4	10.6	10.3	9.9
			Jan. 1, 1941	3,532	211.4	164.8	17.8	14.3	12.9	12.1	----
California	3	50 [1]	Aug. 6, 1939	1,444	183.7	139.9	54.4	12.5	11.4	11.1	10.6
			Jan. 1, 1941	1,570	161.1	136.9	15.7	11.9	11.2	10.8	----
Florida	4	90	May 15, 1937	3,365	144.2	119.7	14.9	10.8	10.3	9.8	9.2
			Jan. 1, 1941	3,646	147.3	132.1	13.3	11.9	11.3	10.9	10.3

[1] State routes. Others are U S numbered routes.

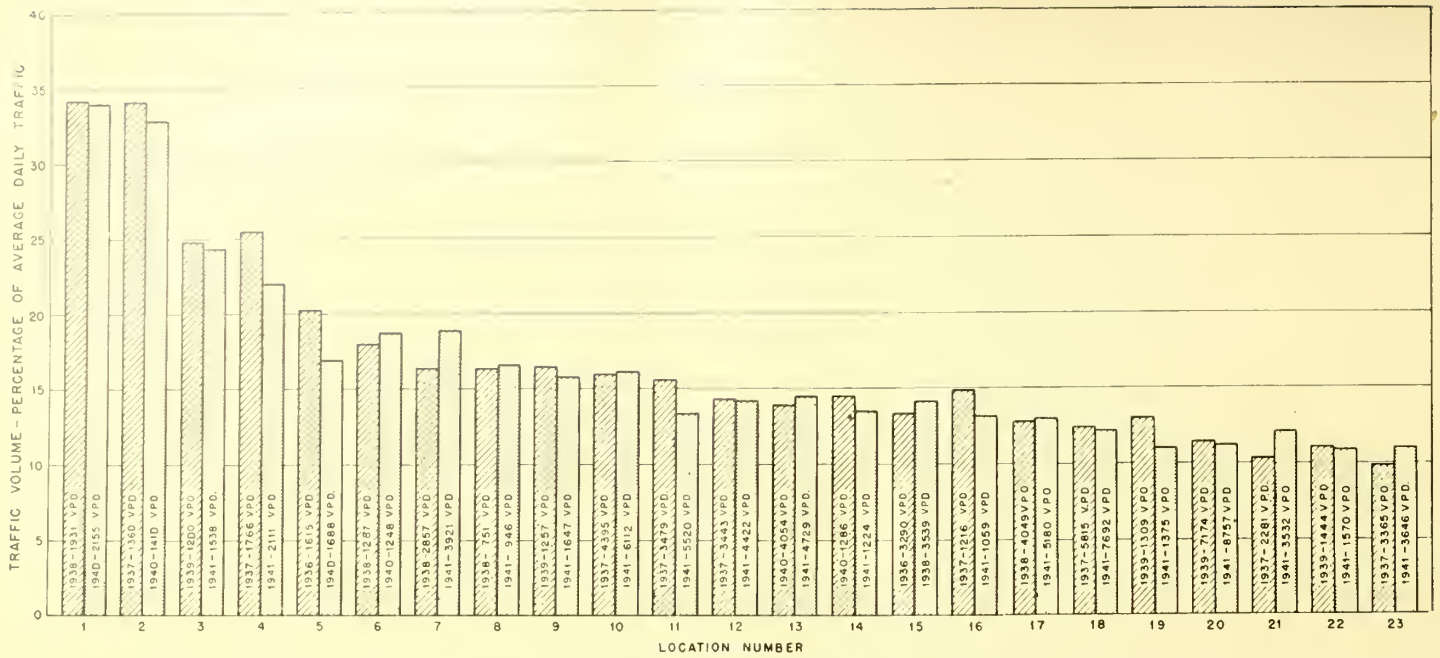


Figure 52.—Comparison of thirtieth highest hourly volumes for two different years at idential locations.

thirtieth highest hour percentages for different years at the same location.

The fact that there is this invariability in the thirtieth highest hour at a given location adds greatly to its worth as a design criterion, since with a given annual average daily volume for some future year obtained through forecasts, it is at once possible to compute with considerable confidence the traffic load on the facility during the thirtieth highest hour of that future year. For example, if conditions indicate that a 20-percent rise in the annual average may be expected in 10 years, a similar 20-percent increase should be expected in the thirtieth highest hour; that is, if the facility is able to handle that much traffic. If it is not, severe congestion is apt to result and traffic will either use other routes or the drivers will be subject to long delays.

If the facility is already partially congested during 30 hours of a year, a 20-percent increase in total traffic will worsen the congestion far more than this proportionate amount. A 20-percent gain in annual average daily traffic will increase the number of hours of congestion from 30 to approximately 100 hours, a gain of 333 percent. Furthermore, the average congestion during the 100 hours will be much more than during the 30 hours of the previous year. Thus at an average location, if the average daily volume is 19,600 vehicles the thirtieth highest hour is 3,000 vehicles per hour. Should traffic increase 20 percent during the next 10 years, or to 23,520 vehicles per day, the thirtieth highest hour during that year will be higher in direct proportion—specifically, 3,600 vehicles per hour—if, as previously stated, the facility is capable of handling this larger volume. A volume of 3,000 vehicles per hour will then be 12.5 percent of the average daily traffic which, as may be seen from figure 50, is exceeded during about 100 hours of the year.

Table 25.—*Variations in traffic flow on major urban facilities during 1 year*

City and location	Type[1] of facility	24-hour volume			Percentage of average 24-hour volume in certain hourly volumes during year				
		Average for year	Percentage of average in—		Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
			Maximum 24 hours	Tenth highest 24 hours					
		Vehicles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Birmingham, Ala.: Roebuck Blvd.	A–O	6,742	155.9	143.7	17.2	15.5	14.7	13.8	9.9
Chicago, Ill.:									
Leif Erikson Dr.	E–I	41,590	137.9	129.0	12.6	11.2	10.7	10.4	10.1
Michigan Ave.	A–D	69,736	131.9	118.2	10.0	8.9	8.3	8.2	7.9
Monroe St.	A–D	32,102	140.0	124.1	11.5	9.6	9.2	8.9	8.6
Ashland Blvd.	A–I	16,919	129.0	114.6	10.4	9.9	9.6	9.5	9.3
Jackson Blvd.	A–I	20,939	133.3	122.1	11.3	10.2	9.8	9.6	9.4
Sacramento Blvd.	A–I	13,243	142.5	122.5	13.5	12.3	11.9	11.8	11.5
Warren and Washington Blvds.	A–I	39,374	138.4	123.8	12.9	12.1	11.3	10.3	9.9
Lake Shore Dr.	E–I	85 698	140.7	124.6	13.5	11.5	10.9	10.7	10.3
Detroit, Mich.:									
Joy Rd.	I	10,784	139.6	129.9	15.2	11.8	11.3	11.1	10.6
Six Mile Rd.		22,768	124.5	117.3	11.1	9.8	9.8	9.7	9.4
14th St. at Edison		12,894	140.6	122.6	15.3	12.4	12.0	11.8	11.7
Albuquerque, N. Mex.: North 4th St.	A–O	3,375	173.7	146.2	15.2	13.7	13.2	12.8	12.4
Santa Fe, N. Mex.: Don Gaspar St.	A–D	4,679	166.1	133.7	18.6	12.6	12.1	11.8	11.3
New York, N. Y.: George Washington Bridge.	E–O	22,000	245.6	212.4	22.5	18.8	17.8	16.9	11.6
Philadelphia, Pa.:									
Chestnut St. Bridge[2]	E–D	30,200	129.7	120.1	8.7	8.1	8.0	7.3	6.9
Parkway and 22d St.	A–D	51,200	113.6	112.4	11.5	11.4	11.1	11.0	10.8
Spring Garden Bridge	A–I	19,500	122.5	118.2	16.6	12.7	12.4	11.7	11.0
Girard Ave. Bridge	A–I	43,800	120.2	117.1	11.1	10.6	10.2	10.0	9.8
Wissahicken and Ridge Sts.	A–O	40,500	119.1	114.0	11.9	10.4	10.1	9.6	9.4
City Line Bridge	E–O	24,360	148.4	138.7	13.1	11.7	11.4	10.7	10.2
Allegheny and Hunting Park	A–O	29,500	129.9	118.8	10.6	10.1	10.0	9.7	9.4
Broad, Glenwood, Cambria	A–I	51,000	122.3	118.8	9.3	9.2	9.0	8.8	8.6
5th and Roosevelt Blvd.	A–O	23,400	151.1	145.4	16.6	14.3	13.6	13.6	13.3
Ogontz and Olney Ave.	A–O	19,670	124.2	120.9	12.9	10.7	10.5	10.3	10.1
Washington, D. C.:									
Fourteenth St. Bridge	A–I	41,300	138.5	115.7	9.6	9.2	9.0	8.8	8.2
Memorial Bridge	E–I	36,700	151.8	116.9	14.7	13.0	12.4	12.1	11.9
Key Bridge	A–I	32,600	143.0	117.6	11.3	9.7	9.7	9.4	8.8
Anacostia Bridge	A–O	32,278	114.0	104.4	9.0	8.3	8.1	7.9	7.3
Benning Rd. NE.	A–O	27,725	141.6	115.6	12.8	9.1	8.7	8.5	7.5
Bladensburg Rd. NE	A–O	27,123	138.6	107.9	10.4	9.7	9.1	9.0	8.5
Connecticut Ave.	A–I	26,842	116.7	110.0	9.1	8.8	8.5	8.0	7.2
Pennsylvania Ave.	A–I	24,388	123.1	112.4	8.8	8.3	7.9	7.7	6.6
Georgia Ave. NW.	A–O	21,628	125.7	117.1	10.4	9.1	8.7	8.7	8.1
Wisconsin Ave.	A–O	20,786	129.4	111.8	10.4	10.2	9.7	9.5	9.2
Rhode Island Ave. NW.	A–I	19,695	117.6	103.8	9.3	8.8	8.5	8.3	7.9
13th St. NW	C–D	16,857	121.7	106.9	10.6	9.6	9.4	9.1	8.1
K St. NW	A–D	15,618	115.2	109.1	10.7	10.4	10 0	9.9	8.9
Total		28,329	136.8	122.6	12.4	10.9	10.5	10.2	9.5

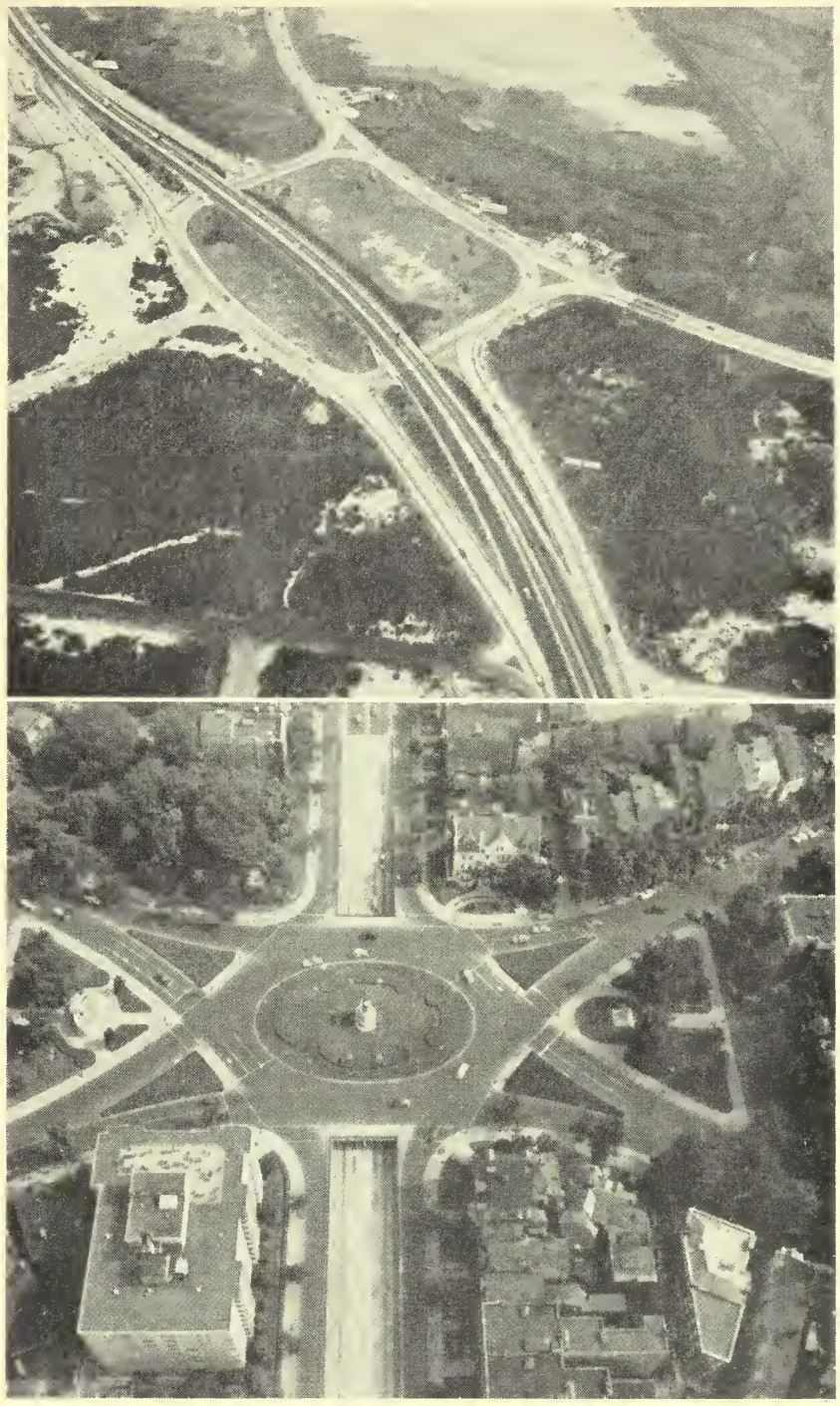
[1] Type of facility code:
E = Expressway. O = Outlying.
A = Arterial. I = Intermediate.
C = City Street. D = Downtown.

[2] One-way.

Peak-Hour Traffic in Urban Areas

Though most of the foregoing has dealt with the conditions on rural highways, more and more facts similar to those presented in table 25 are becoming available for urban facilities. More comprehensive counts are needed, however, before anything like a complete presentation of the relationships between peak and average-hour volumes can be made for city conditions. The data in table 25 do show, nevertheless, that peak hours on the average city facility are a somewhat lower percentage of the annual average 24-hour traffic volume than is the case for rural highways. It should be noted that there are numerous exceptions in the table to this generalization, principally on those streets or highways that serve suburban or outlying areas. While there are insufficient data presently available to permit full explanation of this difference between the thirtieth peak hour relationship in downtown and rural areas, a fact which may well be considered is that **many downtown facilities are severely congested during hundreds of hours of the year.** It is on this type of facility that traffic counters are generally installed. **Peak-hour volumes that otherwise might be recorded several times during the year are impossible of attainment because the facility is incapable of accommodating traffic in excess of the load imposed upon it almost every day during the hours of heaviest demand.** The exceptionally heavy peak load, therefore, must be distributed over alternate routes that are usually more circuitous and less desirable in some other respects than the arterial, were it not congested.

If proposed urban improvements are to provide satisfactory service, the percentage of the average daily traffic used for the design peak hour should be ascertained by a study of facilities that are capable of absorbing some overload during all hours of the year. Arterial streets of this kind are very rare, indeed, under present-day conditions. Until such time as more comprehensive data become available, a percentage for traffic moving in the thirtieth highest hour approximating or only slightly lower than the percentage observed in rural areas is recommended for the design of urban facilities.



Traffic circles take many forms: Shown here are tangent and circular sections for weaving. When traffic circles are no longer able to accommodate their traffic loads the necessary remedial measures, such as an added overpass (above) or tunnel (below), may cost several million dollars.

Effect of Directional Distribution of Traffic

From the discussions in parts III and IV it will be recalled that the capacities of two- and three-lane roads must be in terms of total traffic. The one-direction capacity of these facilities is dependent upon the number of vehicles approaching from the opposite direction, and the hour of the day when the two-directional movement is at a maximum constitutes the period of greatest congestion. Hence the foregoing discussion relating to the percentage of the average daily traffic in the thirtieth highest hour has taken account only of the total traffic moving in both directions. On all multilane facilities, however, and on two- and three-lane roads where improvement to the multilane type is contemplated, consideration of the directional traffic load may be of superior importance. For example, each lane of a four-lane divided facility may be capable of handling 1,500 vehicles per hour, a nominal total of 6,000 vehicles per hour. However, peak-hour traffic flows are not evenly balanced by direction except in very rare instances. A frequent condition on rural highways, for example, is a peak flow of two-thirds in one direction and one-third in the other direction. Consequently, the facility in question would carry only a total of 4,500 vehicles when the capacity of the heavy direction is completely absorbed. Obviously, more than four lanes would be needed to accommodate a total flow of 6,000 vehicles per hour under these circumstances.

As an aid when considering the variations in traffic flow by direction on multilane roadways, tables 26 and 27 show the percentage that the thirtieth highest hour volume is of the annual average 24-hour traffic volume in one and in both directions for a number of rural and urban facilities. Of more significance than the actual percentages for the few rural and urban facilities included in these tables is the relation between the one-direction

Table 26.—*Comparison of variations in total traffic flow with variations in one direction of travel*

	Recorder station No.	Route No.	Year starting	24-hour volume			Percentage of average 24-hour volume in certain hourly volumes during year				
				Average for year	Percentage of average in—		Maximum hour	Tenth highest hour	Twentieth highest hour	Thirtieth highest hour	Fiftieth highest hour
					Maximum 24 hours	Tenth highest 24 hours					
BOTH DIRECTIONS COMBINED:				*Vehicles*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
New Jersey (Essex County)	NB and SB	25	Jan. 1, 1941	62,250	143.6	131.6	11.2	9.6	9.3	9.3	8.8
New Jersey (Edison Bridge)	NB and SB	35	do	25,381	284.3	231.6	24.0	19.1	17.3	16.2	15.4
New Jersey (Woodbridge)	NB and SB	35	do	22,052	229.2	211.5	26.8	18.0	17.0	15.9	13.6
Connecticut	6 and 7	1	Mar. 31, 1939	13,624	292.5	227.9	25.7	21.2	19.2	18.6	17.7
Delaware	C	13	June 8, 1941	6,711	254.2	206.5	23.8	20.5	20.0	19.1	16.9
Wisconsin	2 and 3	[1] 41	Jan. 8, 1938	5,674	305.9	220.2	29.1	22.2	18.9	18.1	16.3
ONE DIRECTION:											
New Jersey (Essex County)	SB	25	Jan. 1, 1941	30,150	159.9	137.8	13.3	11.4	11.1	10.9	10.3
New Jersey (Edison Bridge)	SB	35	do	11,656	436.3	315.3	36.0	28.0	24.7	24.7	23.6
New Jersey (Woodbridge)	SB	35	do	10,766	489.0	255.3	41.5	30.1	26.2	21.7	18.9
Connecticut	6-WB	[2] 1	Mar. 31, 1939	6,813	287.7	244.3	29.7	25.6	24.0	23.4	22.0
Delaware	C-NB	13	June 8, 1941	3,371	344.9	235.9	40.1	34.6	31.4	29.2	25.6
Wisconsin	2-WB	[1] 41	Jan. 8, 1938	2,817	405.6	241.8	43.7	30.7	25.6	22.9	20.2

[1] U. S numbered routes. Others are State routes.
[2] Merritt Parkway.

and two-direction percentages. It will be seen that there are but few cases where the percentage of the average daily traffic moving in one direction during the thirtieth highest hour of the year is not substantially higher than the percentage for both directions. One reason for this, of course, is that the peak volumes in one direction do not always occur at the same time as the peak total volumes. The latter are more apt to occur on multilane facilities when there is a fairly high traffic movement in both directions.

As a practical matter, it is much more likely that the engineer would have information only on total traffic movement, especially for two- or three-lane roads. To illustrate the manner in which the information from table 27 might be employed, let it be assumed that the present annual average daily traffic on a two-lane rural road is 4,800 vehicles per day

Table 27.—*Thirtieth highest hour traffic volume during the year as a percentage of the annual average 24-hour traffic volume in one and in both directions on multilane rural and urban highways and streets*

Location	Percentage of annual average daily traffic in thirtieth highest hour of year	
	Both directions	One direction
RURAL FACILITIES:		
Connecticut (Merritt Parkway)	18.6	23.4
New Jersey:		
U S 1 near Newark	9.3	10.9
Edison Bridge, State Route 35	16.2	24.7
Woodbridge St., Route 35	15.9	21.7
Delaware	19.1	29.2
Wisconsin	18.1	22.9
Rural averages	16.2	22.1
URBAN FACILITIES:		
Chicago (Parkways)[1]	9.9	14.1
New York (George Washington Bridge)	16.9	24.9
Philadelphia [2]	10.6	13.1
Washington, D. C.[3]	9.0	11.9
Urban averages	11.6	16.0

[1] 8 locations.
[2] 9 locations.
[3] 13 locations.

and, in that locality, the thirtieth highest hourly volume of the year on roads that are not badly congested is 16.2 percent of the average daily traffic.

Assume also that it is desired to calculate the capacity of the road that will be needed 10 years hence when the estimated traffic for the design year, including the immediate increase on the improved facility, will be 150 percent of 4,800 or 7,200 vehicles per day. The thirtieth highest hourly volume will be 16.2 percent of 7,200 vehicles, or 1,166 vehicles per hour. This exceeds the practical capacity of a two-lane road even with the best alinement. Furthermore, sight-distance restrictions and other limitations imposed by the terrain might lower the practical capacity of a two-lane road at this location to some value considerably below 900 passenger cars per hour, the practical capacity of a good two-lane road. The required capacity for one direction of a four-lane road would be 22.1 percent (table 27) of 3,600 vehicles, or 796 vehicles per hour. This same value may be approximated by assuming that two-thirds of the thirtieth highest hourly volume of 1,166 vehicles will be traveling in the one direction.

Although conditions differ rather widely among facilities, **it is usually sound, in the absence of detailed information as to traffic volume balances, to assume that two-thirds of the traffic will be in one direction during the design peak hour in rural areas and in outlying urban areas. As the central business districts of large cities are approached, the distribution of traffic by directions usually becomes more evenly balanced, sometimes closely approaching a 50–50 ratio in the central business area.**

A tabulation of traffic by direction of movement for the streets reported in the intersection study shows that in downtown areas an average of 55 percent of the traffic on the congested streets was moving in the heavier direction, and that for 70 percent of these streets between 50 and 55 percent was in this direction of heavier movement. The average directional distribution of traffic on streets in intermediate areas was 61.6 percent in the heavier direction during peak hours with 38.4 percent in the direction of lesser movement. In outlying areas the distribution was 65.6 percent in the heavier direction and 34.4 percent in the other direction.

Traffic During Various Periods of the Day

The information shown in table 28 will be of some assistance to an engineer considering a proposed improvement when the only available traffic data consist of a few short counts. In some cases, by relating the available counts to the information shown in tables 22–27, it will be possible to obtain a fairly accurate estimate of the practical capacity that should be provided or the excess capacity available on an existing facility. **Such a procedure is not recommended, however, and should not be condoned for any project involving extensive improvements when it is still possible to obtain complete traffic information before preparing final plans for the improvements.**

On main rural highways, an average of 74 percent of the day's traffic occurs between 7 a. m. and 7 p. m., with at least 70 percent of the locations represented coming within 5 percent of this value. Local routes in rural areas have a slightly higher percentage of the total day's traffic during this period than the main routes, whereas most urban facilities carry less than 70 percent of their traffic between 7 a. m. and 7 p. m.

Though the 2 p. m. to 10 p. m. and 8 a. m. to 4 p. m. periods each account for approximately half of the total day's traffic on rural routes, the 2 p. m. to 10 p. m. period is definitely the heavier part of the day in urban areas.

The traffic volume during the 8 hours (not necessarily consecutive) that have the highest total flow is of special interest in connection with urban problems since this base is commonly used in making counts for traffic signals and similar improvements. As an illustration of the need for this type of information, the recently revised Manual on Uniform Traffic Control Devices specifies that the total vehicular volume entering an intersection from

Table 28.—*Percentage of traffic during various periods of the day*

Type of highway and location	Percentage of 24-hour volume in the period—											
	6 a. m. to 2 p. m.		2 p. m. to 10 p. m.		10 p. m. to 6 a. m.		8 a. m. to 4 p. m.		7 a. m. to 7 p. m.		Highest 8 hours (not consecutive)	
	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range
RURAL CONDITIONS: [1]												
Main rural highways	41	38–44	49	46–52	10	8–13	49	44–55	74	69–79	57	53–60
Local rural routes	45	41–50	48	44–50	7	5– 9	52	50–58	78	75–82	59	53–65
URBAN CONDITIONS: [2]												
Washington, D. C. (weekdays):												
13 locations	40	38–42	47	46–50	13	10–15	41	36–47	70	65–74	52	49–60
In-bound	45	40–56	43	39–46	12	9–15	44	37–51	70	64–76	53	50–62
Out-bound	34	38–37	52	48–60	14	11–17	38	32–43	69	63–73	53	48–60
Chicago Parkways (weekdays):												
8 locations	38	34–46	48	45–49	16	12–21	44	38–49	68	61–72	55	50–61
In-bound	49	35–54	39	30–44	12	9–16	55	49–60	74	67–79	58	52–63
Out-bound	25	15–35	56	48–62	19	13–26	33	21–45	64	55–72	58	50–65
Detroit, Mich.: 3 locations	39	38–41	48	47–50	12	11–14	40	38–42	70	67–72	54	53–55
Delaware River Bridge:												
Maximum weekday	35	----	44	----	21	----	40	----	62	----	49	----
To Philadelphia	38	----	37	----	25	----	41	----	59	----	49	----
To Camden	31	----	51	----	18	----	39	----	65	----	55	----
Maximum Sunday	26	----	46	----	28	----	32	----	50	----	52	----
To Philadelphia	9	----	55	----	36	----	14	----	32	----	72	----
To Camden	37	----	35	----	18	----	54	----	71	----	54	----
New York City bridges and tunnels:												
Weekdays	--	----	--	----	--	----	--	----	71	----	--	----
Saturdays	--	----	--	----	--	----	--	----	67	----	--	----
Sundays	--	----	--	----	--	----	--	----	64	----	--	----

[1] Range includes the averages for at least 70 percent of the locations
[2] Range includes the averages for all locations.

all approaches must average at least 750 vehicles per hour for any 8 hours of an average day before the vehicular volume alone will warrant a fixed-time signal in an urban area. For rural areas, the corresponding criterion is 500 vehicles per hour.

Table 28 shows that 57 percent of the day's traffic on main rural highways and about 53 percent of the day's traffic at urban locations occur during the highest 8 hours of the average day. A fixed-time traffic signal, therefore, would not be warranted by the traffic volume alone unless the average annual volume from all approaches to the intersection exceeded 11,300 vehicles per day at an urban location, or 7,000 vehicles per day at a rural location.

Based on the average yearly traffic patterns for rural roads and urban facilities, as shown by tables 22 and 24, these annual volumes correspond to the peak hourly volumes shown in table 29.

Based on the results of the intersection studies included in part v of this report, and assuming the usual condition that two-thirds of the traffic on each highway will be moving in one direction during the peak hours, the total traffic through an intersection with one approach on each road loaded to its possible capacity will be as shown in table 30. With proper signal timing, the total intersection volume will be independent of the distribution of traffic between the two roads.

It is therefore apparent that the traffic volume at the intersection of two two-lane rural roads or at the intersection of two 40-foot city streets with parking must approach and in some cases exceed the possible capacities of the average intersection of the same type with traffic signals before the traffic volume warrants alone would justify the installation of traffic signals. It is also apparent that fixed-time signals on two-lane rural highways cannot be justified by the traffic volume warrant until the volume of traffic on such roads at times exceeds their practical capacities, especially when one of the intersecting roads carries about 75 percent of the total traffic at the intersection. These are important considerations in view of the part that traffic signals play in increasing the capacity of intersections at grade by helping to eliminate confusion and serious accidents.

Table 29.—*Annual average 24-hour volume and corresponding peak hourly volumes warranting a traffic signal, as based on Nation-wide traffic pattern and traffic-signal warrant from the Manual on Uniform Traffic Control Devices*

	Total traffic volume from all approaches at—	
	Urban intersection	Rural intersection
Annual average 24-hour volume	11,300	7,000
Peak hourly volumes:		
Highest hour	1,402	1,744
Tenth highest hour	1,232	1,246
Thirtieth highest hour	1,152	1,072
Fiftieth highest hour	1,074	988

Table 30.—*Total hourly volume through intersections of various types when one approach on each of the intersecting roads is operating at its possible capacity*

Type of intersection	Total hourly intersection volume with one approach on each road operating at its possible capacity, based on—	
	Possible capacity of average intersection of the same type	Highest capacities observed
Two two-lane rural roads	1,500	1,800
Two 40-foot city streets, with parking	1,125	1,800
Two 40-foot city streets, without parking	2,140	3,750

The Financing of Highways by Counties and Local Rural Governments, 1931–1941

The results of an extensive long-term study by the Bureau of Public Roads are reported in a new publication, *The Financing of Highways by Counties and Local Rural Governments, 1931–1941.* The report presents a discussion, and detailed statistical data, concerning the financing of highways by county and local rural governments during the 11-year period. Included is information for each year, by States, on county and local receipts, expenditures, and debt for rural highways, which has long been in demand but never available heretofore.

The publication was made possible by the collection of the basic data through the intensive effort of the State highway departments, the county and local governments, and the field offices of the Bureau of Public Roads; and the analysis and presentation were the work of the Financial and Administrative Research Branch, Bureau of Public Roads.

The publication is for sale, at 45 cents a copy, by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., to whom all orders should be sent. Prepayment is required. The Bureau of Public Roads cannot undertake free distribution of the publication.

Bibliography on Roadside Control

The Bureau of Public Roads has recently issued a processed publication, *Bibliography on Roadside Control,* in which are included a general reference section on the title subject and special sections on outdoor advertising, set-back regulations, and roadside zoning. The listings are arranged by States, and chronologically beginning with 1930. A brief digest of the cited reference accompanies each listing.

The bibliography is intended to serve as an aid to highway planners and administrators in their activities dealing with the problem of roadside control. It was compiled by the Land Studies Section, Financial and Administrative Research Branch of the Bureau.

The publication is available to highway engineers and administrators for official use, and may be obtained by those so qualified by writing to the Bureau of Public Roads, Washington 25, D. C.

U. S. GOVERNMENT PRINTING OFFICE, 1949

Axle-Load and Gross-Load Trends

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported [1] by JOHN T. LYNCH, Chief, Planning Surveys Section and T. B. DIMMICK, Head, Current Data Analysis Unit

East of the Rockies, about one-third of the commercial vehicles (exclusive of light trucks) are three- and four-axle tractor-truck semitrailer combinations. They carry two-thirds of the tonnage hauled, and are responsible for almost all of the heavy axle loads.

IN the period beginning toward the close of World War I, much was said about the damage being done to improved highways by the heavily laden trucks that were then coming into general use. For example, in the First Biennial Report of the California Highway Commission, issued in 1918, we find this statement:

> "The present State highways are being subjected to constant abuse by too heavily loaded trucks and other agencies. The statute books of California already contain sufficient legislation to regulate and penalize these violations, but the delinquency lies in the enforcement of these laws."

However, a survey made in California [2] in 1920 did not reveal any loads that we would regard as heavy today. Of a total of 219 motortrucks weighed at five locations, there were only 33 with a capacity of 3 tons or more, and the average gross weight of these was 14,640 pounds. The average rear axle weight was only 9,889 pounds, but the load was generally on solid tires, which were more damaging to the highway than are the pneumatic tires of today.

In surveys made in Vermont,[3] New Hampshire,[4] and Ohio,[5] in 1926 and 1927, no gross weights were found as great as 25,000 pounds, and no axle loads as great as 18,000 pounds.

In a 1926 survey in Connecticut,[6] however, almost 1 percent of the 82,738 trucks observed weighed over 25,000 pounds gross, and they were so loaded that the average rear axle weight was 18,500 pounds. There was one gross load of 30,400 pounds with a rear axle weight of 25,500 pounds.

A survey in Cook County, Ill.,[7] (adjacent to the city of Chicago) in 1925, revealed some surprisingly heavy axle loads. At 5 of the 17 stations operated, rear axles weighing 17,500 pounds or more were found on from 2 to 14 percent of the loaded vehicles. There were several axle loads over 30,000 pounds and one as high as 36,000 pounds and these loads were principally on solid tires. In the majority of these cases the load was brick or cement. Figure 1 shows a heavy truck used at that time.

Some 30 years ago there was great concern about the damage being done to improved highways by what were then considered as heavily laden trucks. With the replacement of solid rubber by pneumatic tires, the introduction of legal limitation of sizes and weights of vehicles, resulting in wider use of vehicle combinations with multiple axles, and the construction of thicker pavements in the great road-building era beginning in the 1920's, the earlier concern was forgotten. By 1931 the loads carried on the highways were rarely heavy enough to overtax their structural capacity.

The State-wide highway planning surveys provided the first Nation-wide data on frequency of heavy loads for the 1936–37 period, and have annually collected such data beginning in 1942. The information, presented in this article, shows how rapidly the frequencies of heavy loads have increased in recent years. In the 1936–37 period, gross loads of 40,000 pounds or more amounted to only about 1 percent of the total number of commercial vehicles on the highways. By 1942 this frequency had tripled, and by 1948 had again doubled. Frequency of heavy axle loads shows a similar rise. But, since the number of trucks has also increased greatly, the total number of heavy axle loads found on the highways in 1948 was eighteen times the number in the 1936–37 period, and the rate of increase appears to be accelerating rapidly.

A study of heavy load frequencies by regions indicates that the most favorable situation exists in the Western regions, where legal limitation of 50 feet or more in length permits advantageous distribution of heavy loads on vehicle combinations with five or more axles. In the remainder of the country, where the length is limited to 45 feet in most States, vehicles with more than four axles are little used, and three- and four-axle combinations carry the bulk of the highway tonnage. It is these two types that have the highest frequency of heavy axle loads.

Legal limitation of axle loads, with effective enforcement, seems to be the only way our pavements can be protected. Somewhat less-stringent length limits would encourage wider use of vehicle combinations that spread the load over a sufficient number of axles, thus permitting reasonable increases in pay loads

[1] This paper was presented at the twenty-ninth annual meeting of the Highway Research Board in Washington, D. C., December 16, 1949.

[2] *Report of a Study of the California Highway System*, by the U. S. Bureau of Public Roads, to the California Highway Commission and Highway Engineer; 1920.

[3] *Report of a Survey of Transportation on the State Highways of Vermont*, by the Bureau of Public Roads, U. S. Department of Agriculture, and the Vermont State Highway Department, 1927.

[4] *Report of a Survey of Transportation on the State Highways of New Hampshire*, by the Bureau of Public Roads, U. S. Department of Agriculture, and the New Hampshire State Highway Department, 1927.

[5] *Report of a Survey of Transportation on the State Highway System of Ohio*, by the Bureau of Public Roads, U. S. Department of Agriculture, and the Ohio Department of Highways and Public Works, 1927.

[6] *Report of a Survey of Transportation on the State Highway System of Connecticut*, by the Bureau of Public Roads, U. S. Department of Agriculture, and the Connecticut State Highway Department, 1926.

[7] *Report of a Study of Highway Traffic and the Highway System of Cook County, Illinois*, by the U. S. Bureau of Public Roads and the Cook County Highway Department, 1925.

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Figure 1.—A typical heavy truck circa 1920.

It seems probable that the stations in the Cook County survey were so located as to intercept exceptionally heavy hauling, for Mr. Frank T. Sheets, Chief Highway Engineer for Illinois, said in February 1926 that the percentage of trucks around Chicago which exceeded the legal axle-load limit of 16,000 pounds was small.[8] Certainly there is no evidence that heavy loads comparable to those weighed in Cook County were to be found in appreciable numbers outside the Chicago area.

Regulation of Sizes and Weights

Laws governing the sizes and weights of motor vehicles were in effect in most States at that time. Fifteen States passed such laws prior to 1919, and by 1927 all of the States except Kansas and Montana had legal limitations on sizes and weights.[9] Kansas passed its first limitation act in 1929 and Montana in 1931. In most States the laws have been changed many times since they were first enacted, but the weight limits have remained largely within the ranges of 8,000–9,500 pounds per wheel for States limiting wheel loads and 16,000–22,400 pounds per axle for States limiting axle loads. The cumulative effect of these laws over the years has been to promote increasingly wider use of vehicle combinations with the load spread over a number of axles.

In addition to limiting weights, fees were increased for vehicles with solid tires, thus hastening the change-over to pneumatics and lessening road damage—for it was the impact of solid tires on the macadam roads and on the narrow concrete pavements with relatively thin edges prevailing at that time that was principally responsible for the damage during the first World War and in the years immediately following.[10] Many of the early concrete roads had uniform slab thicknesses of only 6 inches or even 4 inches. Following the Bates Road Test in 1921, thickened edges of 7 inches, 8 inches, and 9 inches became prevalent. More recently, slabs of uniform thickness of 8 inches and 9 inches have been constructed by many States;[11] and several States, including New Jersey, where heavy axle loads are particularly frequent, have used a 10-inch uniform thickness.

Thus, the problem of heavy loads was attacked from three angles: legal limitation, vehicle design, and pavement design. By 1931 the loads carried on the highways were rarely heavy enough to overtax their structural capacity, and Commissioner MacDonald made the following statement (which has been quoted recently by those interested in obtaining higher axle-load limits): "The roads are more destroyed really by climatic and soil conditions than they are by any use that is made of them." How greatly heavy axle-load frequencies of today exceed those prevailing at the time that statement was made will be shown in the data presented in this paper.

First Comprehensive Data Obtained in 1936–37

Our present large investment in roadway pavements makes it impracticable for us to provide for greater loads by increasing pavement thicknesses on any but a small fraction of our improved road mileage. There remain then only vehicle regulation and vehicle design as effective means of protecting our roadway plant from excessive loads.

The State-wide highway planning surveys, which were started in 1935, gave for the first time Nation-wide comprehensive data from which accurate determination could be made of the frequency of occurrence of heavy gross loads and heavy axle loads on our highways.[12] Nearly all of the States conducted truck-weighing operations in 1936 or 1937, although a few did not do so until a year or two later.

Figure 2 shows the situation at that time with regard to heavy gross loads. The heights of the bars indicate the number of gross loads of 40,000 pounds or more per 1,000 commercial vehicles on the highways, including both loaded and empties, in each of the census regions. In the Pacific region, 47 out of each 1,000 vehicles passing the weighing stations had a gross load of 40,000 pounds or more. In the remainder of the United States, loads as heavy as 40,000 pounds were comparatively infrequent and amounted to more than 10 per 1,000 vehicles, or 1 percent, in only three regions—the Middle Atlantic, the New England, and the East North Central.

[8] *Traffic control and the regulation of overloading of motor-trucks*, by Frank T. Sheets; Proceedings of the Twelfth Annual Conference of Highway Engineering, University of Michigan, 1926, p. 85.

[9] *Federal Regulation of the Sizes and Weight of Motor Vehicles*, report of the Interstate Commerce Commission, Aug. 14, 1941. House Doc. No. 354, 77th Cong., 1st Sess., pp. 63–66.

[10] *The highways of the country and the burden they must carry*, a symposium of eight articles; PUBLIC ROADS, vol. 1, No. 2, June 1918.

[11] *Design details of structural features of concrete pavements as practiced by the various State highway departments in 1946*, by T. J. Kauer; Report of Committee on Concrete Pavement Design, American Road Builders' Association Technical Bulletin No. 121, 1947; pp. 48–49. Also, *Comments on concrete road design*, Engineering-News Record, Nov. 15, 1934; pp. 618–619.

[12] *Amount and characteristics of trucking on rural roads*, by J. T. Lynch and T. B. Dimmick; PUBLIC ROADS, vol. 23, No. 9, July-Aug.-Sept. 1943.

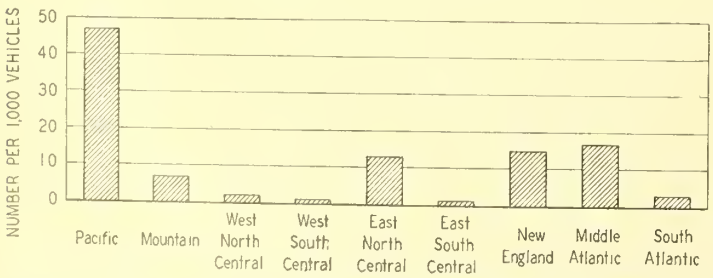


Figure 2.—Frequency of gross loads of 40,000 pounds or more, per 1,000 vehicles, by regions, in 1936–37.

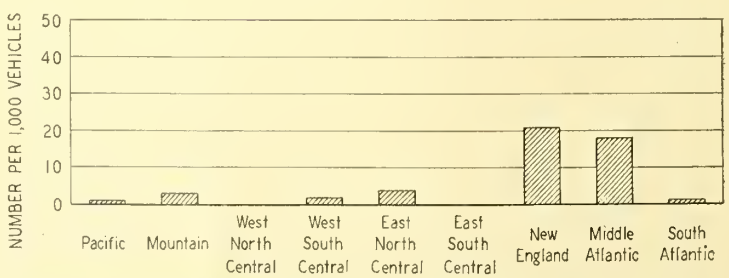


Figure 3.—Frequency of axle loads of 20,000 pounds or more, per 1,000 vehicles, by regions, in 1936–37.

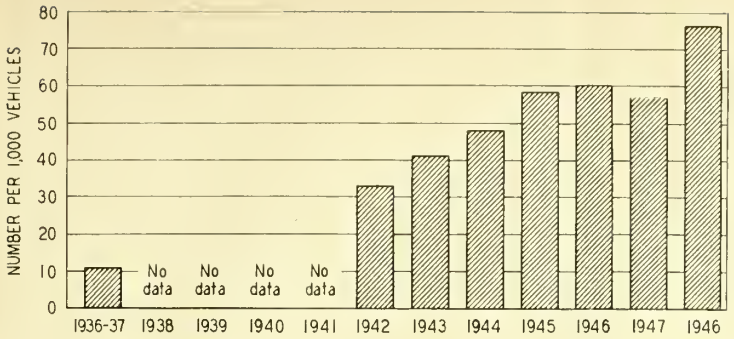


Figure 4.—Frequency of gross loads of 40,000 pounds or more, per 1,000 vehicles, by years.

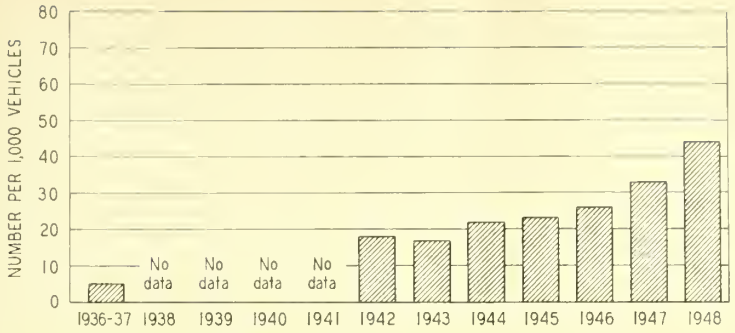


Figure 5.—Frequency of axle loads of 20,000 pounds or more, per 1,000 vehicles, by years.

Figure 3 shows the frequency of occurrenee of heavy axle loads of 20,000 pounds or more in the different regions at the time of the 1936–37 surveys. Only in the New England and Middle Atlantic regions were axle loads as great as 20,000 pounds found in appreciable numbers—the frequency of axle loads of this magnitude in these two regions being approximately 20 for each 1,000 commercial vehicles, loaded and empty.

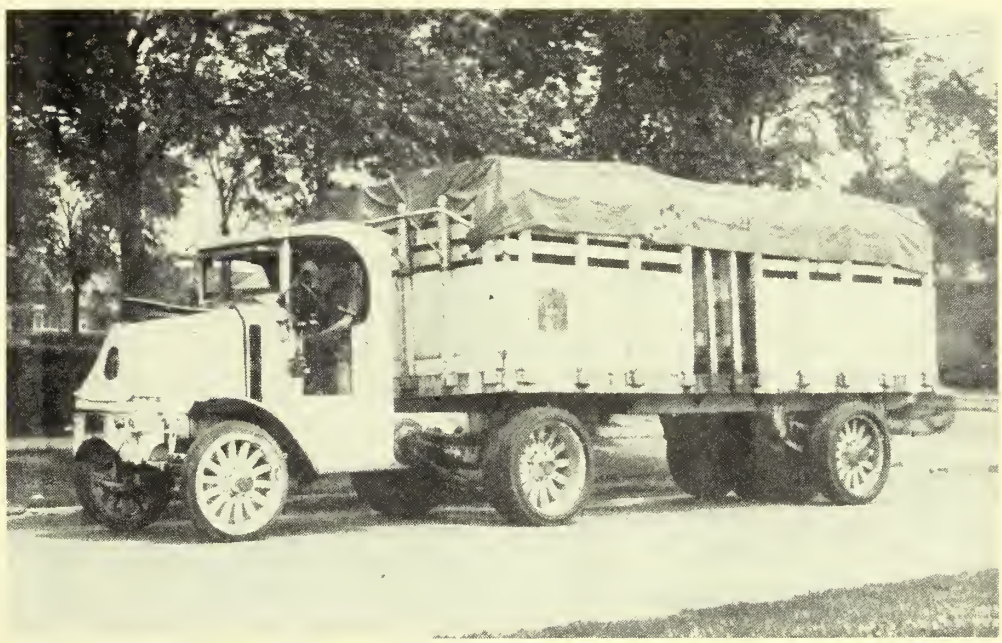
Following the original collection of weight data by the highway planning surveys in the 1936–37 period, only fragmentary data conecrning truck weights were obtained until 1942, when the increased loading because of World War II activities began to cause some concern. In that year and in each year since then data which give a clear indication of trends have been obtained at a number of representative stations, operated for 8 hours in the summer in practically all of the States. The number of trucks weighed each year in these repeat surveys has ranged from about 50,000 to over 100,000.

Increase in Frequency of Heavy Gross Loads and Heavy Axle Loads

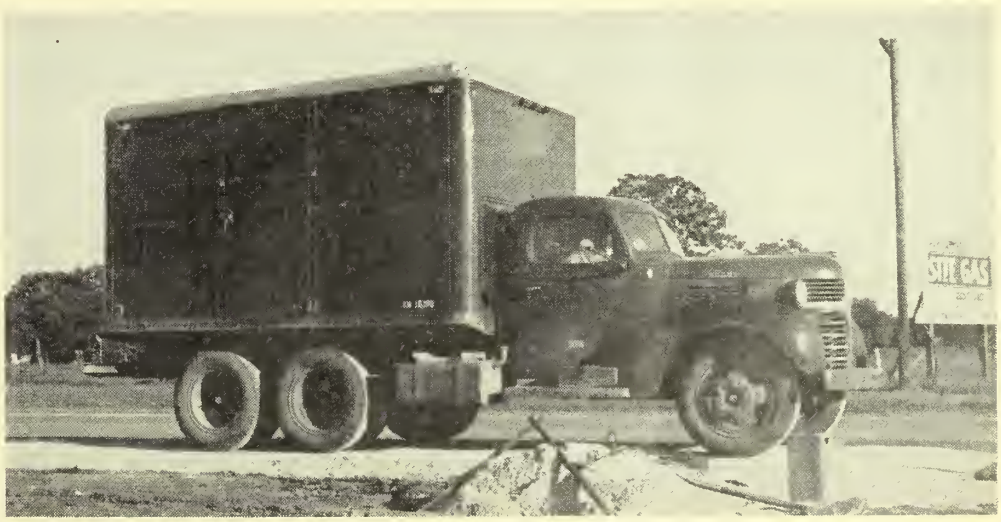
Figure 4 shows the steady increase in frequency of heavy gross loads since the 1936–37 surveys. In 1942, the first year of the war, gross loads of 40,000 pounds or more were about three times as frequent as in the prewar period. At the close of the war the frequency of gross loads did not decline (except slightly in 1947) but continued to increase, so that in 1948 gross loads of 40,000 pounds or more were more than twice as frequent as in 1942, and seven times as frequent as in the 1936–37 period.

The long-range tendeney over the years has been to spread the load over more axles, thus permitting large increases in gross loads without corresponding increases in axle loads. At the time of the early surveys (1926–27), mentioned in the first part of this paper, only two-axle vehicles were in general use. Gross load was only from one-fourth to one-half greater than the maximum axle load. In the recent surveys, on the other hand, it has been found that the gross load averaged about twice the maximum axle load for vehicles of all types, and nearly three times the maximum axle load for combinations, which are the vehicles carrying the heaviest loads.

It naturally follows that the frequency of heavy gross loads has increased faster than the frequency of heavy axle loads, over the years. Since 1945, however, this tendency has been reversed, and heavy axle loads are increasing in frequency faster than heavy gross loads. For each 1,000 vehicles passing over the highways there were, in 1945, 5.3 times the number of gross loads of 40,000 pounds or more than there were in the 1936–37 period (fig. 4), compared to 4.6 times the number of axle loads of 20,000 pounds or more found in the earlier period (fig. 5). In the 3 years from 1945 to 1948, however, the heavy axle-load frequency increased 91 percent, compared to an increase of only 32 percent for the heavy gross load frequency. The reason for this reversal ean most likely be



A 1924-vintage tractor-semitrailer, probably one of the earliest of this type. Note the center wheels on the rear axle.



A dual-tire, three-axle truck.

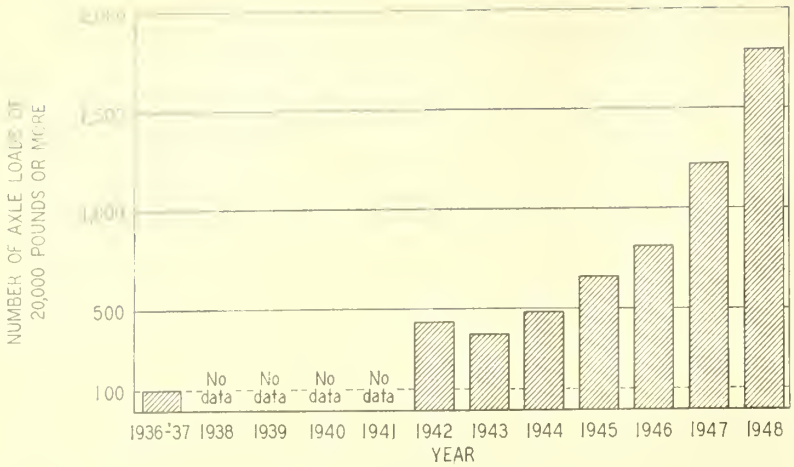


Figure 6.—Relative total number of axle loads of 20,000 pounds or more, 1942–48, using the number in 1936–37 as an index of 100.

found in legal length limitations, in effect over much of the country, which effectively discourage the use of vehicles with five or more axles. While, in earlier years, the increased gross loads were spread over an increasing number of axles, this tendency has been stopped at four axles in many States and increases in gross loads can now only be accomplished by increases in axle loads.

In 1948, the number of axle loads of 20,000 pounds or more in each 1,000 vehicles passing over the highways was approximately 9 times the number in the 1936–37 period. This tells only part of the story, however, because the number of trucks on the highways almost doubled during this time. Figure 6 compares the relative number of axle loads of 20,000 pounds or more found on the highways each year from 1942 to 1948 with the number found in the 1936–37 period used as an index of 100. In 1948, our highways were being subjected to approximately 18 times as many axle loads of 20,000 pounds or more as they were in 1936–37. It is plain, therefore, that Mr. MacDonald's statement made in 1931 does not apply to the conditions we find today. The alarming thing about the picture presented in figure 6 is that the rate of increase of the absolute number of heavy axle loads appears to be accelerating rapidly. In the 2-year period from 1946 to 1948, the number of axle loads in excess of 20,000 pounds more than doubled, which is a greater rate of increase than was found in any previous 2-year period.

Regional Variation in Frequency of Heavy Loads

Figure 7 shows the frequency of gross loads of 40,000 pounds or more in 1948, by regions. The Pacific region still had the highest frequency of loads of this magnitude, but the use of heavy vehicles in the East North Central and Middle Atlantic regions had increased to such an extent that the frequency of occurrence of gross loads of 40,000 pounds or more was greater than 100 in each 1,000 vehicles—not much below the frequency of 127 per 1,000 vehicles in the Pacific region.

Figure 8 shows the frequency of heavy axle loads in 1948, by regions. Data from all of the States except Pennsylvania, Florida, Wyoming, and Oregon are included. The Middle Atlantic region had by far the greatest frequency, there being in this region 153 axle loads weighing 20,000 pounds or more for each 1,000 trucks or truck combinations of all types (including panels and pick-ups), both loaded and empty. Since Pennsylvania data are missing for 1948, the height of the bar for the Middle Atlantic region actually reflects conditions in New York and New Jersey.

A three-axle tractor-semitrailer combination.

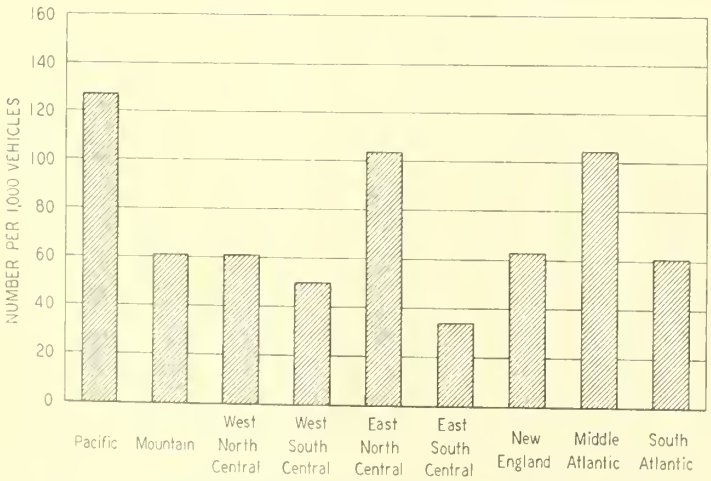


Figure 7.—Frequency of gross loads of 40,000 pounds or more, per 1,000 vehicles, by regions, in 1948.

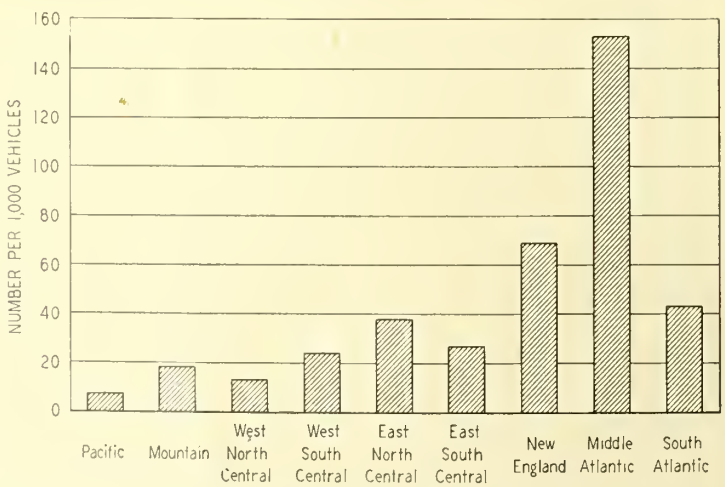
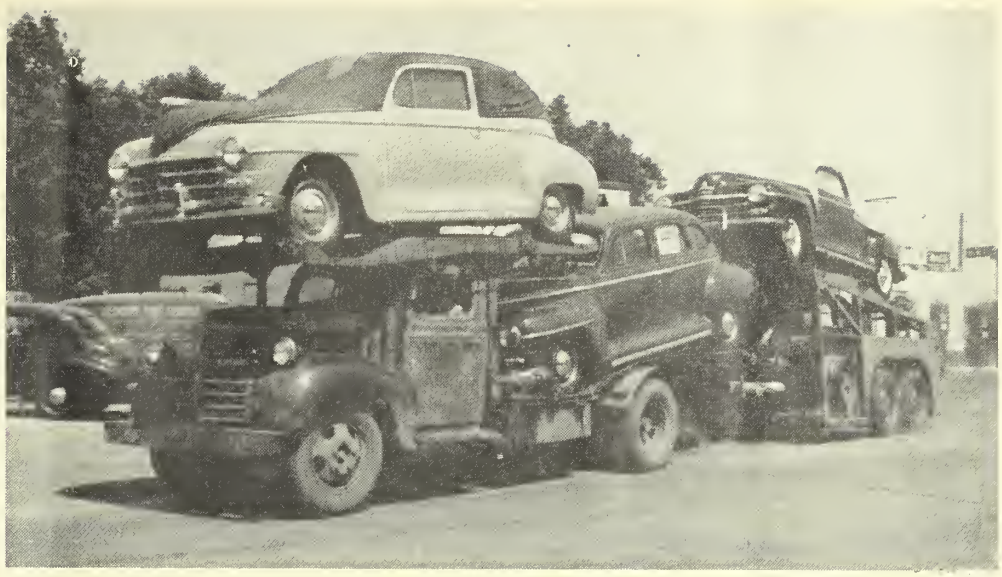


Figure 8.—Frequency of axle loads of 20,000 pounds or more, per 1,000 vehicles, by regions, in 1948.

The laws in these States, as well as in several other Eastern States, have the effect of encouraging heavy axle loads.

New England, with about 69 axle loads of 20,000 pounds or more for each 1,000 vehicles, had the second highest frequency. It will be noted that the Pacific region, which had the greatest frequency of heavy gross loads (fig. 7), had the lowest frequency of heavy axle loads, there being only 7 axle loads as great as 20,000 pounds for each 1,000 vehicles observed in this region.

Figure 9 shows the frequency of heavy axle loads in the 10 States in which axle loads of 20,000 pounds or more occurred more frequently than 50 per 1,000 vehicles, arranged in order of frequency of occurrence. New Jersey, with 239 axle loads of 20,000 pounds or more, of which 165 weighed 22,000 pounds or more, in each 1,000 commercial vehicles, had the greatest frequency. There is no specific axle-load limit in New Jersey, the weight permitted on an axle being determined by the size and number of tires. Evidently the tire criterion does not serve to limit axle loads greatly, since a number of axle loads exceeding 30,000 pounds were recorded in the survey, and there was one axle load of 40,100 pounds.



A four-axle auto transporter.

The nine States having the greatest frequency of heavy axle loads are all along the Atlantic seaboard. The tenth State, Ohio, with an 18,000-pound axle load limit, had 58 axle loads in excess of 20,000 pounds for each 1,000 vehicles observed.

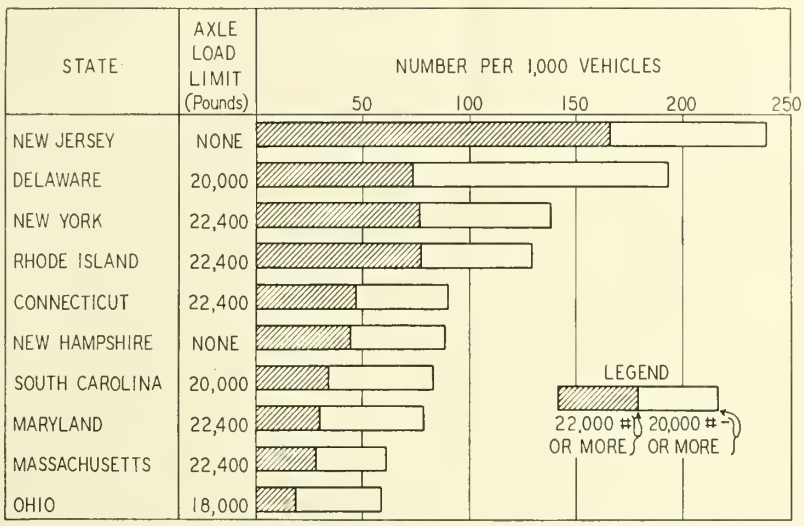


Figure 9.—Frequency of axle loads of 20,000 pounds or more and of 22,000 pounds or more, per 1,000 vehicles, in the 10 States with highest frequencies, in 1948.

Frequency of Heavy Axle Loads Dependent on Types of Vehicles Used

The frequency of occurrence of heavy axle loads in the different States is dependent upon the types of vehicles used, as well as upon the legal axle load limitation. The usage of the different vehicle types, in turn, is dependent to a considerable extent upon the legal length and other limitations. The legal limitations affecting axle loads and vehicle lengths vary throughout the country but are fairly uniform in the Mountain and Pacific regions. In these regions there is an axle load limit of 18,000 pounds throughout and no State has a legal length limit for tractor-truck and semitrailer combinations of less than 50 feet. The 50-foot limit applies in Oregon, and all of the other States in the two regions permit lengths of 60 feet or more. Throughout the remainder of the country most of the States have length limits of 45 feet for tractor-truck and semitrailer combinations, and this limit has a definite bearing on the type of vehicle used.

Figure 10 shows the frequency distribution of vehicles of different types for the Mountain and Pacific regions combined, and for the remainder of the United States. Panels and pick-ups and other light, two-axle, single-tire trucks of similar character are excluded from

A four-axle combination vehicle, widely used throughout the United States.

A five-axle combination vehicle, popular in the western part of the country.

the distribution in both sections of the country the two-axle dual-tire trucks constitute approximately one-half of the total, and in the Mountain and Pacific regions the remaining vehicles are rather evenly distributed among a number of different types, with the five-axle tractor-truck and semitrailer combination predominating. In the Central and Eastern regions the number of vehicles with more than four axles is minute and the three-axle tractor-truck semitrailer combination type constitutes almost one-third of the total.

Although the dual-tire, two-axle, single-unit truck is most frequent in occurrence, it does not carry the greatest pay-load tonnage. This is shown in figure 11, which gives the percentage of the total pay-load tonnage hauled by vehicles of different types. In the Mountain and Pacific regions the five-axle tractor-truck and semitrailer combination carries a higher percentage of the pay-load tonnage than any other type of vehicle, whereas in the remainder of the United States it is the three-axle tractor-truck and semitrailer combination which carries the bulk of the tonnage.

A six-axle tractor-semitrailer combination.

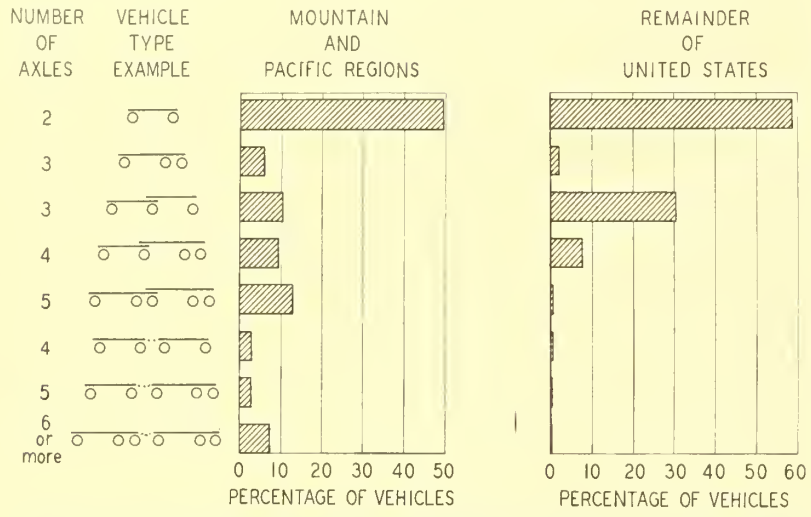


Figure 10.—Percentage distribution of vehicles of various types (exclusive of two-axle, single-tire vehicles) in the West and in the remainder of the United States, in 1948.

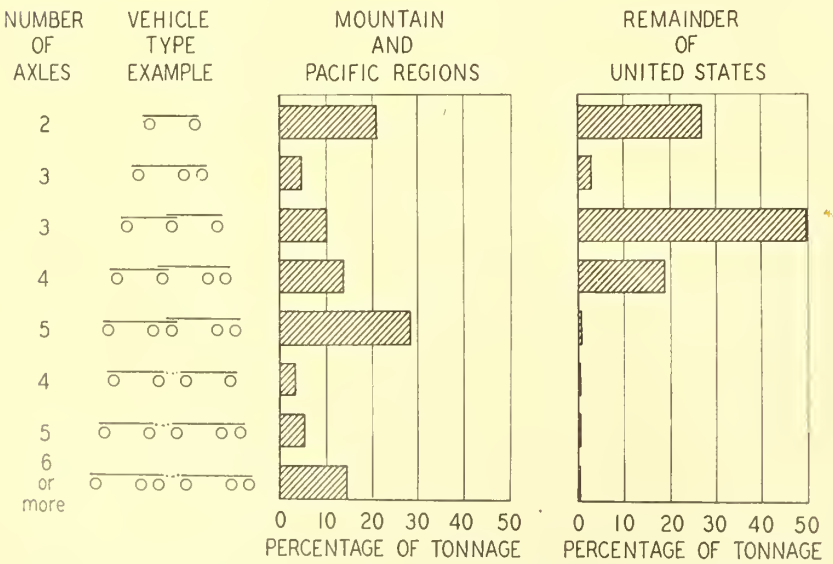


Figure 11.—Proportion of tonnage hauled by vehicles of various types (exclusive of two-axle, single-tire vehicles) in the West and in the remainder of the United States, in 1948.

Figure 12 shows the average gross load and the average pay load for vehicles of different types, for the United States as a whole. The five-axle tractor-truck and semitrailer combination has a slightly higher average pay load than any other type of vehicle, although the group of tractor-truck and full trailer combinations with six or more axles has the highest gross load.

Figure 13 shows the frequency of heavy axle loads for the vehicles of different types. Since there are significant differences between the Eastern and Central regions as regards frequency of heavy axle loads, largely because of differences in State laws, these groups of regions, as well as the Western group, are shown separately in this chart.

In all the regions it is the four-axle tractor-truck and semitrailer combination that has the greatest frequency of heavy axle loads. In the Eastern regions almost half of these vehicles, including both loaded and empties, have an axle load weighing 20,000 pounds or more, whereas in the Western regions less than 10 percent of them have axle loads of this magnitude. Almost invariably it is the rear axle of the tractor that has the heaviest load. It is easy to see why this should be so, because any uniform loading would tend to overload this axle unless the dual axles at the rear of the semitrailer were placed well forward, and such an arrangement would frequently cause violation of the axle-group loading regulations in effect in many States.

In figure 14 the amount of pay load carried is taken into consideration. Here the frequency of axle loads of 20,000 pounds or more per 1,000 tons of pay load is shown. On this basis it is the three-axle tractor-truck and semitrailer combination that has the highest frequency of heavy axle loads in the Eastern and Western regions, while the four-axle tractor-truck and semitrailer combination continues to hold first place in the Central regions.

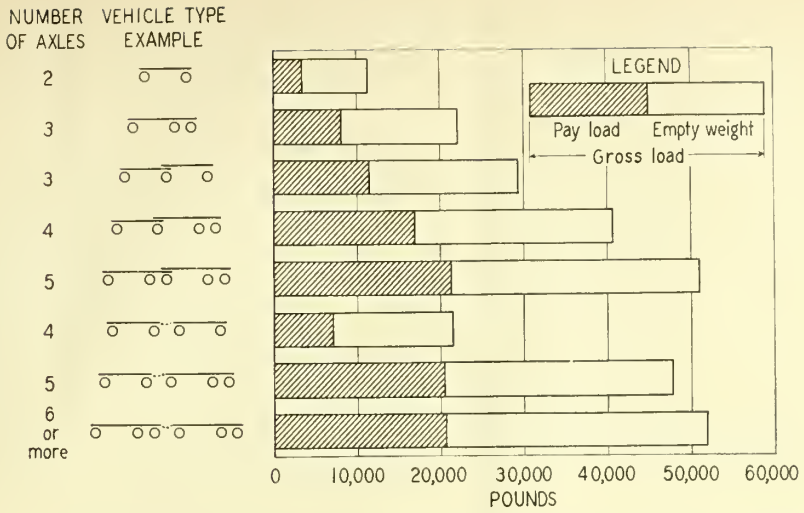


Figure 12.—Average pay load and average gross load of vehicles of various types (exclusive of two-axle, single-tire vehicles) in 1948.

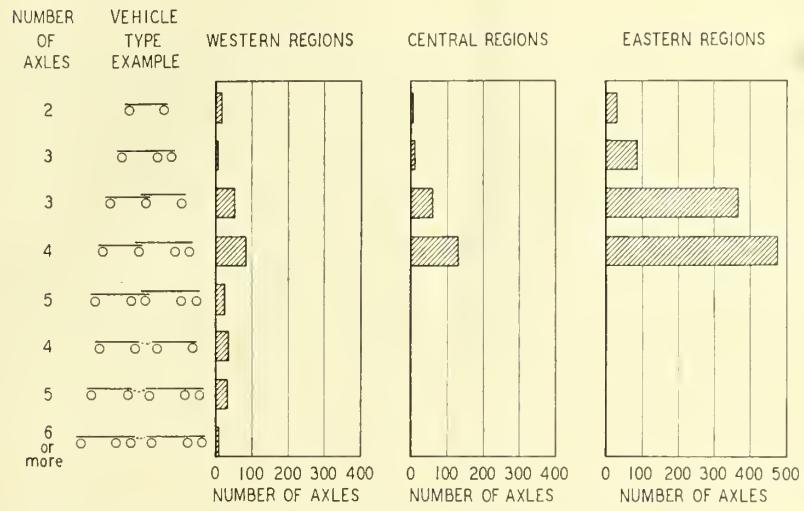


Figure 13.—Frequency of axle loads of 20,000 pounds or more, per 1,000 vehicles, by vehicle types (exclusive of two-axle, single-tire vehicles), in three geographic regions, in 1948.

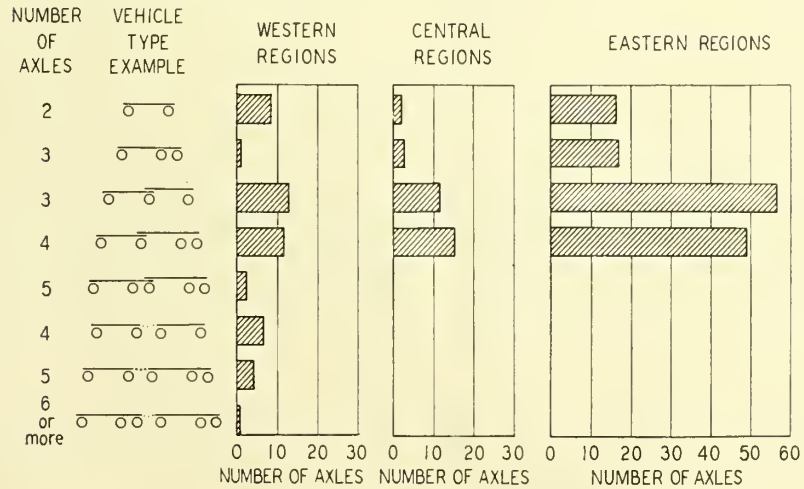


Figure 14.—Frequency of axle loads of 20,000 pounds or more, per 1,000 tons of pay load, by vehicle types (exclusive of two-axle, single-tire vehicles), in three geographic regions, in 1948.

The five-axle tractor-truck and semitrailer combination, which is used in appreciable numbers only in the Mountain and Pacific regions, has the lowest frequency of heavy axle loads per 1,000 tons of pay load of any of the more commonly used vehicle types.

Length Limits Restrict Use of Five-Axle Combination

The failure to make use of this five-axle combination type in the Central and Eastern regions has been attributed by some to shortcomings in operating characteristics of the vehicle design, but any such difficulties do not appear to be sufficiently great to restrict the usage in the West. A more evident reason for the scarcity of this type of vehicle throughout the Central and Eastern States is the 45-foot length limitation for tractor-truck and semitrailer combination that prevails throughout most of this area. The average wheel-base length for a five-axle, tractor-truck and semitrailer combination, over the whole United States, was 43.4 feet from center of front wheel to center of rear wheel. Obviously the over-all length of the vehicle, including the overhang, front and rear, would have to be greater than 45 feet with this length of wheel base. The two-axle tractor-truck with two-axle semitrailer, on the other hand, which is one of the principal offenders as regards heavy axle loads, had an average wheel-base length of 34.1 feet, which permits a vehicle length well within the 45-foot limitation.

The photograph at the bottom of page 283 (*left*) shows a four-axle combination of a type widely used throughout the country. The gross load on this vehicle was 68,900 pounds and the weight on the second axle was 27,690 pounds. This compares with less than 17,000 pounds each for the third and fourth axles. Such unequal loading is not uncommon for vehicles of this type.

The five-axle combination shown in the photograph at the bottom of page 283 (*right*), which was weighed in California, is the most popular type of heavy vehicle in the West. It is 49 feet long and would therefore violate the length limit in effect in 23 of the 37 Central and Eastern States. The gross weight was 66,700 pounds, almost the same as that of the four-axle combination (p. 283, *left*), but the maximum axle load was only 17,500 pounds.

The weights for single axles and axle pairs were as follows:

Axle number	*Single axle (pounds)*	*Axle pair (pounds)*
1	7,600	
2	17,500	29,100
3	11,600	
4	15,000	30,000
5	15,000	

This type of load distribution is typical of that found on many of the five-axle combinations. Apparently the vehicle was so designed and loaded as to take maximum advantage of legal limits in effect in the area throughout which it was operated. The second axle was loaded for maximum traction almost to the 18,000-pound limit. The third axle, probably not a drive axle, carried a lighter load so that the 32,000-pound limit for an axle pair was not exceeded. The load at the rear was equally distributed between the fourth and fifth axles. The axle-group loading, also, was probably just within the legal limit, for the California law permits 59,500 pounds within

a 30-foot length, which is about the probable distance from the center of the second axle to the center of the fifth axle, though there is no record of the actual measurement in this case. The axle group load limit recommended by the American Association of State Highway Officials, however, was doubtless exceeded somewhat, as that limit is 52,650 pounds for a 30-foot length.

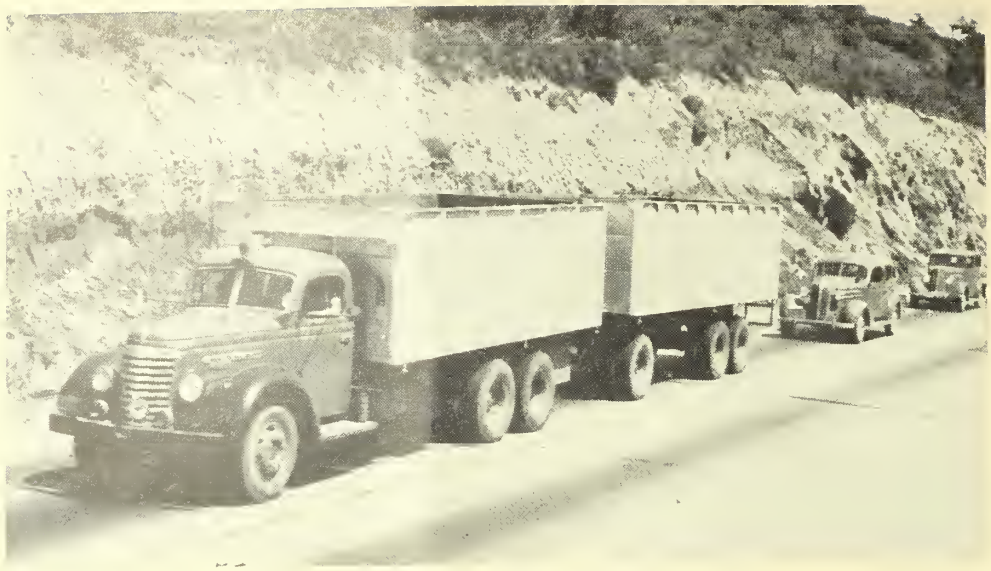
The five-axle semitrailer combinations have a better record of compliance with State weight laws than combinations of other types. In the Pacific region, for example, the percentage exceeding one or more of the weight limits was 9 percent for the five-axle combinations, compared to 12 percent for combinations of other types.

The laws regarding axle-group weights are somewhat more lenient in California and Washington than recommended by the American Association of State Highway Officials, and 22 percent of the five-axle combinations had axle groups loaded beyond the Association's recommended limits. In most cases, however, the excess weight was small, amounting to more than 10 percent excess in the case of only 10 percent of the vehicles.

The number of axles with loads exceeding the 18,000-pound limit per 1,000 vehicles was only 64 for the five-axle combinations compared to 95 for combinations of other types in the Pacific region. As regards compliance with axle-load limits, therefore, the five-axle tractor-truck semitrailer combination has a materially better record than other types of large vehicles.

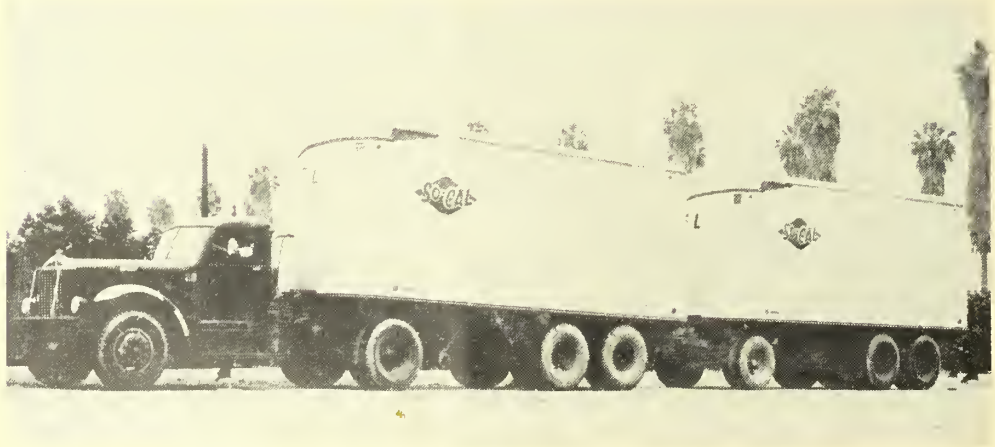
Enforced Axle-Load Limits Needed to Protect Highways

The major part of the Nation's principal highways was constructed in the period from 1920 to 1940. These existing highways must be maintained, and gradually reconstructed or replaced, with the funds currently available for the purpose. It is obviously impossible to meet the rapidly increasing frequency of heavy axle loads by thickening the pavements, as was done to some extent in the earlier years of road building. Legal limitation of axle loads, with effective enforcement, seems to be the only way our pavements can be protected. This does not necessarily mean, however, that gross loads and pay loads may not be allowed to increase, provided the vehicle is so designed as to spread the load over a sufficient number of axles. Somewhat less-stringent length limits, within the A. A. S. H. O. recommendation, would encourage such design and would probably reduce pressure for higher axle-load limits. As a specific example, if those States which have a 45-foot length limit for tractor-truck and semitrailer combination would raise it to 50 feet, as recommended by the American Association of State Highway Officials, this would encourage the widespread use of the type of vehicle which carries more tonnage than any other single type in the West and does so with a very low frequency of excessively heavy axle loads.



A three-axle truck with three-axle full trailer.

A three-axle tractor-semitrailer with two-axle full trailer.



A four-axle tractor-semitrailer with three-axle full trailer.

Traffic Trends on Rural Roads in 1948

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by THOMAS B. DIMMICK, Head, Current Data Analysis Unit

Total travel on rural roads in 1948 broke all records, exceeding the previous year's high by 6 percent and the prewar peak by 17 percent. On the 345,000 miles of main rural roads in the United States travel in 1948 was almost 148 billion vehicle-miles, of which about 77 percent was by passenger cars, 1 percent by busses, and 22 percent by freight-carrying vehicles.

Trucks and combinations hauled 13 percent more ton-mileage of freight in 1948 than in the previous year and 42 percent more than in 1941. These increases in freight carried were made, notwithstanding small drops in the proportion of trucks loaded, because of the rise in use of heavier commercial vehicles, beginning during the war and continued in 1948, although at a reduced rate. Truck-combination travel was 9 percent higher than in the previous year, 68 percent higher than in 1941, and 213 percent higher than in 1936. The average load carried by commercial vehicles in 1948 was 73 percent heavier than in 1936 and about 4 percent above the 1947 load weights.

In 1948, over 5 percent of all trucks and combinations, loaded and empty, exceeded a legal weight limit, and 18 percent of the combinations were overweight in some particular. The highest percentage of overloading was in the East North Central region where 8 percent of the vehicles of all types and 21 percent of the combinations were found to exceed legal weight limits. From 1947 to 1948 the percentage of overweight vehicles increased in all regions except the Pacific and New England regions.

TRAVEL on rural roads in 1948 broke all previous records for the third consecutive year. The 1948 volume of traffic was 6 percent higher than that of 1947 and 17 percent higher than that of 1941, the prewar peak. Regionally the increase over 1947 ranged from 5 percent in the Middle Atlantic and South Atlantic regions to 9 percent in the West North Central region. These facts were established from the records of about 700 automatic traffic recorders operated continuously throughout the year at permanent stations on rural roads, both main and local, in all of the States.

The variation in rural-road travel for three main geographic divisions and in the United States as a whole is illustrated in figure 1 for the year 1948, for 1947, and for 1941, the prewar peak year.

The chart shows that, with minor exceptions, travel in each month in each of the three geographic regions and in the United States as a whole was well above that of the corresponding month of the earlier years. Average monthly travel was higher in 1947 than in 1941 in each of the regions, and still higher in 1948.

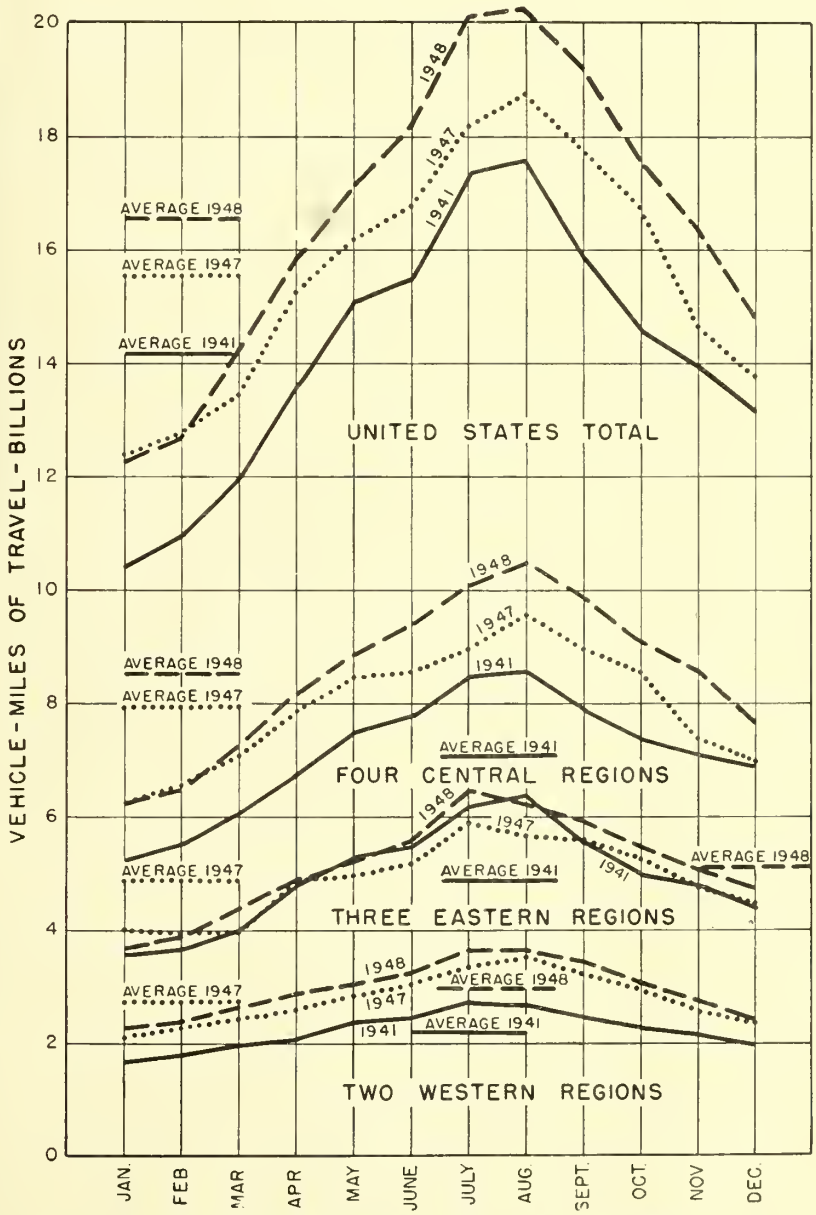


Figure 1.—Travel on all rural roads in 1941, 1947, and 1948, by months.

The summer travel peak, which was greatly diminished during the war, has slowly been regaining, during the postwar years, its former importance in the annual traffic pattern.

The flattening out of the summer traffic peak during the war years and the subsequent recovery of this peak to prewar proportions are shown in the ratios of the July-August

traffic each year to the average 2-month traffic for the year (one-sixth of the total annual traffic):

Year:	
1940	1.23
1941	1.23
1942	1.13
1943	1.13
1944	1.14
1945	1.14
1946	1.17
1947	1.19
1948	1.22

It is not surprising to find that when we entered the war the summer peak traffic dropped from 23 percent above the annual average to only 13 percent above the annual average for 2-month periods. It is somewhat surprising, however, to find that not until 1948 has summer traffic assumed approximately the same relative importance in the annual traffic pattern that it had in the prewar years. Undoubtedly the shortage of automobiles tended to discourage vacation driving in the first 2 years following the close of the war.

1948 Summer Loadometer Survey

The machine counts from which the traffic-volume data were obtained provide no classification by vehicle type, since they record only the total number of vehicles of all types passing. However, trends in type, weight, and characteristics of commercial vehicles were established from data recorded in a summer survey, repeated each year since 1942, in nearly all of the States. In 1948, 544 stations in 44 States were operated at times and under conditions comparable to those existing in earlier surveys. The information obtained in these summer surveys, together with additional information concerning traffic classification collected in a number of States throughout the year, was used to determine the changes that had taken place since the comprehensive Nation-wide surveys of 1936 and 1937 were made.[1]

All States participated in the 1948 survey except Florida, Oregon, Pennsylvania, and Wyoming. In Pennsylvania an expanded weight survey was in progress and in Wyoming one was being completed, but reports from neither were available for inclusion in this analysis.

The majority of the weight stations were operated during July, August, and September. California completed most of the work in June; Texas operated its stations from May through August; Tennessee commenced in May and ended in October; while Minnesota conducted the operations from April to September.

The stations used in these surveys were selected initially to give a representative cross section of traffic on main rural roads, and were operated for one or more 8-hour periods on a weekday, generally from either 6 a. m. to 2 p. m., or from 2 p. m. to 10 p. m. All traffic passing through the stations during the period was counted and classified into the following categories: local passenger cars; foreign (out-of-State) passenger cars; panel and pick-up trucks;[2] other two-axle, four-

Table 1.—*Survey period, number of stations operated, number of vehicles counted, and number weighed in each State in the special weight survey during the summer of 1948*

Region and State	Survey period	Number of stations	Vehicles counted		Trucks and combinations weighed
			All vehicles	Trucks and truck combinations	
New England:					
Connecticut	July 26–Aug. 10	10	28,037	5,494	2,203
Maine	July 26–Aug. 6	10	22,446	3,430	1,673
Massachusetts	Aug. 9–20	10	34,114	5,812	2,077
New Hampshire	Aug. 6–13	5	14,724	1,846	522
Rhode Island	July 26–30	5	12,063	2,331	986
Vermont	Aug. 2–6	5	9,151	742	742
Subtotal		45	120,535	19,655	8,203
Middle Atlantic:					
New Jersey	Aug. 2–19	10	72,065	12,608	2,026
New York	Sept. 20–24	20	39,202	10,762	3,693
Pennsylvania	[1]				
Subtotal		30	111,267	23,370	5,719
South Atlantic:					
Delaware	Aug. 9–18	4	15,189	3,863	745
Florida	[1]				
Georgia	Aug. 16–Sept. 2	6	13,668	3,368	2,290
Maryland	July 26–Aug. 12	9	6,912	1,985	1,093
North Carolina	Aug. 10–31	12	24,083	6,000	3,800
South Carolina	Sept. 8–22	10	16,253	4,350	2,096
Virginia	Aug. 10–23	10	20,600	5,066	1,714
West Virginia	Aug. 10–Sept. 2	9	11,580	3,180	1,359
Subtotal		60	108,285	27,812	13,097
Eastern regions, subtotal		135	340,087	70,837	27,019
East North Central:					
Illinois	Aug. 24–Sept. 7	47	83,658	15,325	6,324
Indiana	Aug. 5–Sept. 1	20	39,099	9,056	3,768
Michigan	July 20–Aug. 7	8	20,560	3,119	1,907
Ohio	July 27–Aug. 12	5	9,062	2,566	677
Wisconsin	Aug. 5–26	12	25,717	3,450	1,951
Subtotal		92	178,096	33,516	14,627
East South Central:					
Alabama	July 14–Aug. 5	10	9,984	2,692	1,782
Kentucky	June 7–July 24	10	11,175	3,012	1,228
Mississippi	Aug. 17–Sept. 7	13	18,153	5,021	3,089
Tennessee	May 26–Oct. 30	10	9,386	2,378	1,455
Subtotal		43	48,698	13,103	7,554
West North Central:					
Iowa	July 26–Aug. 13	10	12,790	2,275	2,180
Kansas	Aug. 5–18	10	9,354	1,967	1,155
Minnesota	Apr. 19–Sept. 8	40	115,839	22,913	10,038
Missouri	Aug. 16–Sept. 3	14	66,067	13,314	7,302
Nebraska	July 22–Aug. 18	20	20,869	4,324	4,262
North Dakota	July 22–Sept. 1	15	19,711	4,349	2,668
South Dakota	July 26–Aug. 13	12	8,076	1,671	1,493
Subtotal		121	252,706	50,813	29,098
West South Central:					
Arkansas	July 9–Aug. 15	10	15,376	4,802	1,350
Louisiana	July 26–Aug. 7	10	9,209	2,742	1,397
Oklahoma	July 7–23	10	13,538	3,454	3,454
Texas	May 18–Aug. 30	17	27,374	6,585	1,799
Subtotal		47	65,497	17,583	8,000
Central regions, subtotal		303	544,997	115,015	59,279
Mountain:					
Arizona	July 26–Aug. 6	10	9,385	1,979	676
Colorado	Aug. 13–27	10	23,939	3,831	715
Idaho	Aug. 2–25	13	13,658	2,533	2,336
Montana	Aug. 4–31	17	15,063	2,649	2,190
Nevada	Aug. 3–17	10	7,188	1,041	836
New Mexico	Aug. 5–Sept. 7	6	6,295	1,383	691
Utah	July 26–Aug. 6	10	14,765	2,878	1,187
Wyoming	[1]				
Subtotal		76	90,293	16,294	8,631
Pacific:					
California	June 2–July 2	20	[2] 67,760	13,620	5,032
Oregon	[1]				
Washington	Sept. 8–21	10	16,926	3,362	2,185
Subtotal		30	84,686	16,982	7,217
Western regions, subtotal		106	174,979	33,276	15,848
United States total		544	1,060,063	219,128	102,146

[1] No survey made.
[2] Passenger cars not counted in California. Figure given is an estimate based on data from other reports.

[1] See *Traffic trends on rural roads in 1947*, by T. B. Dimmick, Public Roads, vol. 25, No. 7, Mar. 1949; *Traffic trends on rural roads in 1946*, by T. B. Dimmick and M. E. Kipp, Public Roads, vol. 25, No. 3, Mar. 1948; *Traffic trends on rural roads in 1945*, by T. B. Dimmick, Public Roads, vol. 24, No. 10, Oct.-Nov.-Dec. 1946; and *Amount and characteristics of trucking on rural roads*, by J. T. Lynch and T. B. Dimmick, Public Roads, vol. 23, No. 9, July-Aug.-Sept. 1943.

[2] Single-unit trucks with a carrying capacity of less than 1½ tons.

tire trucks; two-axle, six-tire trucks; three-axle trucks; tractor-semitrailer combinations; truck and trailer or tractor-semitrailer and trailer combinations; and busses. The combination-type vehicles were further subdivided according to the number of axles of each.

The survey period, number of stations operated, number of vehicles counted, and number weighed are shown for each State in table 1. Over a million vehicles were counted at all stations during the period of the survey. About one-fifth of the vehicles counted were freight-carrying vehicles, of which almost one-half were weighed.

Wherever traffic volume permitted, all trucks and truck combinations were stopped and weighed. Where this procedure was impracticable, a sample was obtained by weighing all of the less common types and omitting only vehicles of types sufficiently common to establish their characteristics from a sample. The type of vehicle, whether loaded or empty, the number of axles, and weight of each axle were recorded. The axle-spacing and total wheel-base length of the heavier vehicles [3] were measured. Passenger cars and busses were counted but not stopped for weighing.

Prewar Travel Trend Fulfilled

Figure 2 shows in chart form the vehicle-mileage of travel on all rural roads, by type of vehicle, for each year from 1936 to 1948, inclusive. It is apparent that the effect of the war on the long-term upward trend in traffic volumes has now been entirely overcome. A straight line from the top of the bar for 1936 to the top of the bar for 1948 fits very closely the prewar trend from 1936 to 1940. The bar for 1941 extends well above this line, but it is generally recognized that 1941 was a year of exceptional activity in preparation for the war.

In the case of travel of trucks and truck combinations,[4] the 1948 value fits the 1936 to 1940 trend, projected, almost exactly. For truck combinations alone, the 1936 to 1948 line lies above the tops of the bars for all intervening years, indicating an accelerating growth in traffic by vehicles of this type.

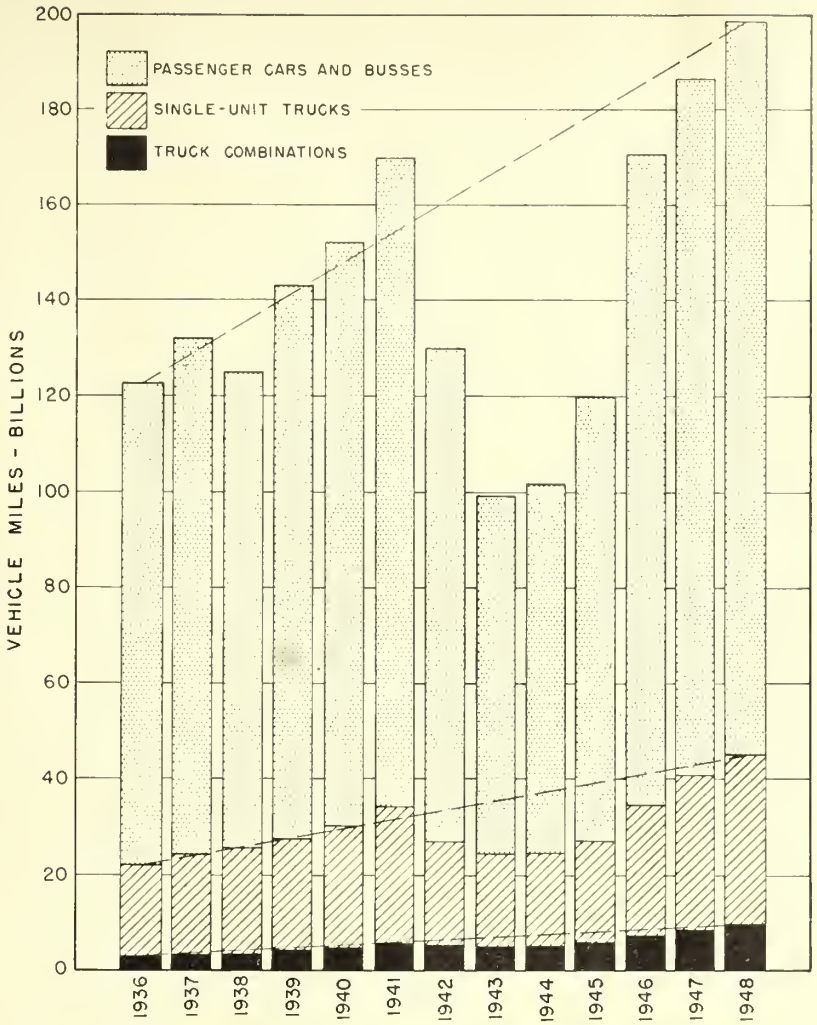


Figure 2.—Travel on all rural roads, 1936–48, by classes of vehicles.

The ratios of traffic volumes on main rural roads in 1948 to those in corresponding months of 1947 are shown in table 2. Main roads, totaling 345,000 miles, are considered to be those of the entire State system in most States, and those of the primary State system in such States as Virginia and Pennsylvania where all or a large part of the rural-road mileage is under State control. The consistent increase in travel by most types of vehicles and in all sections of the country is clearly shown. Passenger-car travel as a whole and that of trucks and truck combinations increased in all regions without exception. There were slight declines in some regions in travel of busses and foreign passenger cars, separately considered.

The percentage distribution of travel in 1948 is given for each census region and for the United States as a whole in table 3. In this table the single-unit trucks are divided into the four classification types based on axle and tire arrangement, while the truck combinations are divided according to the total number of axles of the combination. The classification of vehicles into these types, which has been used only in the last two annual surveys, permits more positive identification than the use of the old "light, medium,

[3] Trucks and truck combinations weighing 13 tons or more and, in addition, single-unit trucks having an axle weighing 18,000 pounds or more.

[4] In this article, the term *truck* is used to indicate a single-unit vehicle; *truck combination* to indicate tractor-truck semitrailer (with or without full trailer) and truck with full trailer; and *trucks and truck combinations* to indicate all of these vehicles together.

Table 2.—*Ratio of 1948 traffic on main rural roads to corresponding traffic in 1947* [1]

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	1.08	1.00	1.04	1.03	1.09	1.23	1.15	1.08	1.12	1.10	1.08	1.08	1.08
Foreign	1.03	1.14	1.08	1.09	1.02	.97	.90	.95	.97	1.03	.97	1.00	1.02
All passenger cars	1.07	1.02	1.05	1.04	1.07	1.15	1.09	1.05	1.08	1.06	1.06	1.06	1.07
Trucks and truck combinations:													
Single-unit trucks	1.07	1.17	1.05	1.09	1.07	1.10	1.14	1.22	1.14	1.11	1.08	1.10	1.12
Truck combinations	1.16	1.07	1.04	1.06	1.11	1.15	1.17	1.17	1.16	1.16	1.03	1.05	1.09
All trucks and truck combinations	1.09	1.14	1.04	1.09	1.10	1.11	1.15	1.20	1.14	1.12	1.05	1.08	1.11
Busses	1.11	.88	.85	.90	1.00	1.05	.95	1.14	1.03	1.10	.99	1.03	.98
All vehicles	1.07	1.05	1.05	1.05	1.07	1.14	1.10	1.09	1.09	1.08	1.05	1.06	1.07

[1] The ratios for "all vehicles" are based on year-round automatic recorder data, while those for the individual vehicle types are based principally on the summer counts.

Table 3.—Percentage distribution of traffic, by vehicle types, in the summer of 1948

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	58.34	60.40	57.62	58.79	60.00	58.76	61.00	57.71	59.53	41.37	69.93	60.28	59.43
Foreign	23.87	16.51	17.79	18.17	21.84	18.54	14.79	13.90	17.75	34.64	10.55	18.69	18.05
All passenger cars	82.21	76.91	75.41	76.96	81.84	77.30	75.79	71.61	77.28	76.01	80.48	78.97	77.48
Single-unit trucks:													
Panel and pickup	5.37	5.16	6.86	5.99	3.95	6.65	6.02	10.28	6.34	10.31	4.22	6.28	6.22
Other 2-axle, 4-tire	.69	.91	1.14	.99	.26	.66	1.10	.63	.61	.38	1.09	.85	.77
Other 2-axle, 6-tire	6.48	10.10	8.70	8.92	6.50	9.63	10.84	9.18	8.62	7.53	5.85	6.41	8.32
Three-axle	.30	.56	.34	.42	.21	.12	.22	.20	.20	.42	.90	.74	.37
	12.84	16.73	17.04	16.32	10.92	17.06	18.18	20.29	15.77	18.64	12.06	14.28	15.68
Tractor-truck and semitrailer combinations:													
3-axle	3.37	4.81	5.47	4.92	4.33	3.64	3.27	5.57	4.28	1.80	1.08	1.32	3.95
4-axle	.10	.62	.73	.60	1.74	.43	1.66	1.40	1.47	1.07	1.28	1.21	1.14
5-axle (or more)	(1)	.01	(1)	(1)	.22	.02	.08	.05	.12	.65	2.16	1.65	.36
All tractor-truck semitrailer combinations	3.47	5.44	6.20	5.52	6.29	4.09	5.01	7.02	5.87	3.52	4.52	4.18	5.45
Truck and trailer combinations:													
4-axle (or less)	.01	.02	.02	.02	.10	------	.17	.14	.11	.22	.46	.38	.13
5-axle	(1)	(1)	----	(1)	.10	------	.01	.01	.05	.21	.40	.33	.08
6-axle (or more)	----	----	----	----	.05	------	(1)	------	.02	.27	1.24	.92	.18
All truck and trailer combinations	.01	.02	.02	.02	.25	------	.18	.15	.18	.70	2.10	1.63	.39
All combinations	3.48	5.46	6.22	5.54	6.54	4.09	5.19	7.17	6.05	4.22	6.62	5.81	5.84
All trucks and truck combinations	16.32	22.19	23.26	21.86	17.46	21.15	23.37	27.46	21.82	22.86	18.68	20.09	21.52
Busses	1.47	.90	1.33	1.18	.70	1.55	.84	.93	.90	1.13	.84	.94	1.00
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

[1] Less than 0.005 percent.

and heavy" categories and, at the same time, the groups are more homogeneous than those formerly used.

The figures in table 3 indicate that truck travel on a percentage basis is heaviest in the West South Central region and only slightly lighter in the West North Central, South Atlantic, and Mountain regions, all of which have comparatively light total traffic. Other areas as, for instance, the New England, East North Central, and Pacific regions, where total traffic generally is heavy, have the smallest percentages of trucks. Certain types of vehicles, apparently favored by local conditions, are found principally in certain regions. For instance, the five-axle (or more) tractor-truck semitrailer combinations and the six-axle (or more) truck and trailer combinations are confined principally to the Western regions. The absence, on the other hand, of certain types in some regions as, for example, the truck and trailer combinations in the East South Central region, is noteworthy.

Average Weights Again Increasing

Figure 3 shows graphically the average weights of loaded and empty trucks and truck combinations, separately and combined, in each year from 1942 to 1948, inclusive, and in a prewar year, generally 1936 or 1937. The weights of single-unit trucks, both the loaded and the empty, increased each year from the 1936–37 period through 1945, then leveled off and even decreased slightly. At the same time weights of both loaded and empty truck combinations increased each year of the period shown. The increase in average weight of loaded combinations from the 1936–37 period to 1948 was 47 percent, compared to only 14 percent for single-unit trucks. The increase for all trucks and combinations was 54 percent, a figure higher than that for either type separately, because of the increased proportion of combinations in the latter year.

The average weights of the various types of loaded and empty trucks and truck combinations in the summer of 1948 are shown in table 4, for the different regions. This table brings out clearly the important differences that exist in the weight characteristics of the vehicles in the different groups. It will be noted, for example, that for the United States as a whole, the loaded three-axle, single-unit trucks weighed about twice as much as the two-axle, six-tire trucks. Similar differences existed throughout the various classifications. On the other hand, the regional differences in average weight for each of the vehicle types that are common throughout the country are surprisingly small. The extremely low weights of truck and trailer combinations in some regions, particularly the West North Central, indicate a predominance of small, home-made trailers of low capacity.

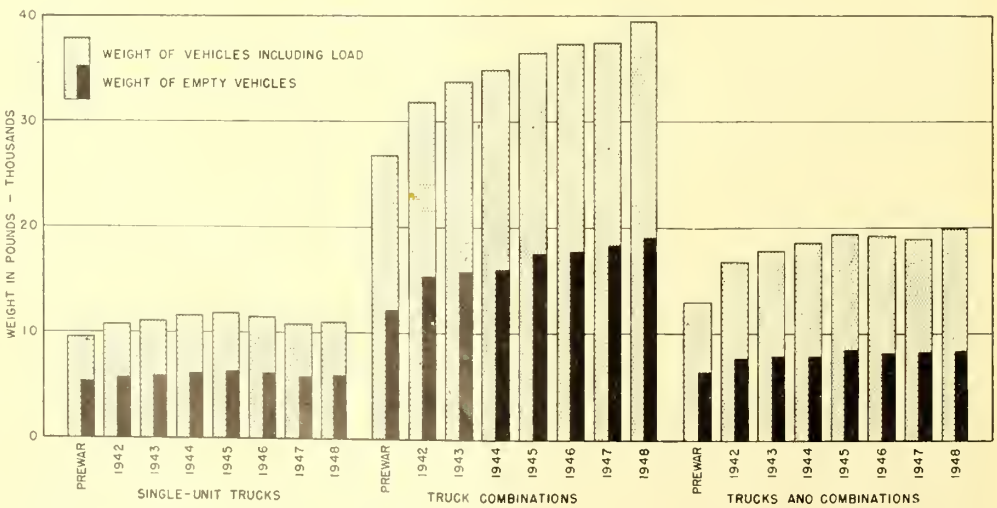


Figure 3.—Average weights of loaded and of empty trucks and truck combinations in the summers of 1942–48 and in a corresponding period of a prewar year.

Table 4.—*Average weights (in pounds) of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1948*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
AVERAGE WEIGHTS OF LOADED VEHICLES													
Single-unit trucks:													
Panel and pick-up	4,619	4,942	4,666	4,760	4,755	5,094	4,909	6,615	5,545	4,967	4,357	4,627	5,143
Other 2-axle, 4-tire	6,321	8,126	9,320	8,668	6,384	6,596	6,730	10,706	7,465	7,542	6,647	6,755	7,788
Other 2-axle, 6-tire	14,290	15,364	13,342	14,299	13,265	14,156	13,191	14,057	13,504	13,471	12,541	12,864	13,679
3-axle	29,102	34,171	23,938	29,076	26,802	28,043	24,804	24,020	25,701	25,303	25,265	25,272	26,700
Average	10,827	13,007	10,914	11,736	10,948	11,792	10,680	10,417	10,808	10,232	10,648	10,501	11,064
Truck combinations:													
Tractor-truck and semitrailer	36,079	42,240	35,482	37,945	37,552	34,640	38,337	35,920	37,028	41,761	45,979	44,888	38,600
Truck and trailer	(1)	(1)	(1)	(1)	56,100	------	13,628	32,164	38,863	55,416	53,659	53,878	50,611
Average	36,087	42,170	35,402	37,883	38,107	34,640	37,513	35,846	37,094	43,942	48,299	47,328	39,455
Average, all trucks and truck combinations	17,541	21,691	18,268	19,502	22,914	18,409	17,932	18,076	19,793	18,999	22,689	21,571	20,034
AVERAGE WEIGHTS OF EMPTY VEHICLES													
Single-unit trucks:													
Panel and pick-up	3,685	4,052	3,703	3,809	3,932	3,891	3,959	4,456	4,139	3,804	3,543	3,705	3,958
Other 2-axle, 4-tire	5,003	4,863	5,188	5,028	4,756	4,798	5,260	6,337	5,425	4,993	4,980	4,983	5,189
Other 2-axle, 6-tire	8,583	8,931	7,186	8,108	7,671	7,493	7,694	7,873	7,712	7,440	7,633	7,542	7,824
3-axle	14,974	13,852	11,856	13,347	13,897	11,687	12,432	16,208	14,042	14,083	14,495	14,417	13,898
Average	6,105	7,067	5,313	6,080	6,022	5,698	6,081	6,157	6,030	5,114	6,089	5,572	5,976
Truck combinations:													
Tractor-truck and semitrailer	19,599	20,405	17,647	18,952	18,956	16,288	19,683	17,260	18,299	21,276	21,588	21,470	18,798
Truck and trailer	------	(1)	(1)	(1)	25,441	------	10,919	19,203	21,070	25,728	26,520	26,357	24,556
Average	19,599	20,411	17,643	18,953	19,413	16,288	19,366	17,300	18,425	22,127	23,341	22,949	19,140
Average, all trucks and truck combinations	8,172	9,455	7,374	8,276	9,864	6,911	8,707	8,477	8,648	7,161	9,516	8,343	8,481

[1] Data omitted because of insufficient sample.

Table 5.—*Comparison of estimated vehicle-miles of travel on main rural roads in 1936, 1941, 1946, 1947, and 1948*

Year	All vehicles, vehicle-miles	Passenger cars and busses[1]		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*		*Millions*
1936	88,412	82.6	73,005	17.4	15,407	82.1	12,650	17.9	2,757
1941	122,505	80.3	98,320	19.7	24,185	78.8	19,057	21.2	5,128
1941: 1936 ratio	*1.39*	*.97*	*1.35*	*1.13*	*1.57*	*.96*	*1.51*	*1.18*	*1.86*
1946	124,149	80.4	99,803	19.6	24,346	73.3	17,838	26.7	6,508
1946: 1941 ratio	*1.01*	*1.00*	*1.02*	*.99*	*1.01*	*.93*	*.94*	*1.26*	*1.27*
1946: 1936 ratio	*1.40*	*.97*	*1.37*	*1.13*	*1.58*	*.89*	*1.41*	*1.49*	*2.36*
1947	137,512	79.2	108,880	20.8	28,632	72.5	20,746	27.5	7,886
1948	147,597	78.5	115,837	21.5	31,760	72.9	23,138	27.1	8,622
1948: 1947 ratio	*1.07*	*.99*	*1.06*	*1.03*	*1.11*	*1.01*	*1.12*	*.99*	*1.09*
1948: 1941 ratio	*1.20*	*.98*	*1.18*	*1.09*	*1.31*	*.93*	*1.21*	*1.28*	*1.68*
1948: 1936 ratio	*1.67*	*.95*	*1.59*	*1.24*	*2.06*	*.89*	*1.83*	*1.51*	*3.13*

[1] Percentages of total 1948 travel by passenger cars and by busses are reported separately in table 3.

Truck Travel Greater in 1948

A comparison of the estimated vehicle-miles of travel on main rural roads is shown in figure 4 for loaded and empty single-unit trucks and truck combinations, separately and combined, for each year from 1936 to 1948, inclusive. This chart further emphasizes the steady growth of truck traffic during the period 1936 to 1941, the temporary effect of the wartime restrictions, and the phenomenal upsurge in highway truck transportation subsequent to the end of the war in 1945.

Table 5 gives a comparison of the estimated vehicle-miles of travel by vehicles of different types on all main rural roads in 1936, the earliest year for which comprehensive weight data are available; in 1941, the peak prewar year, 5 years after the beginning of the surveys; in 1946, 10 years after the beginning of the surveys; and in 1947 and 1948, the latest years. The ratios of 1948 travel to

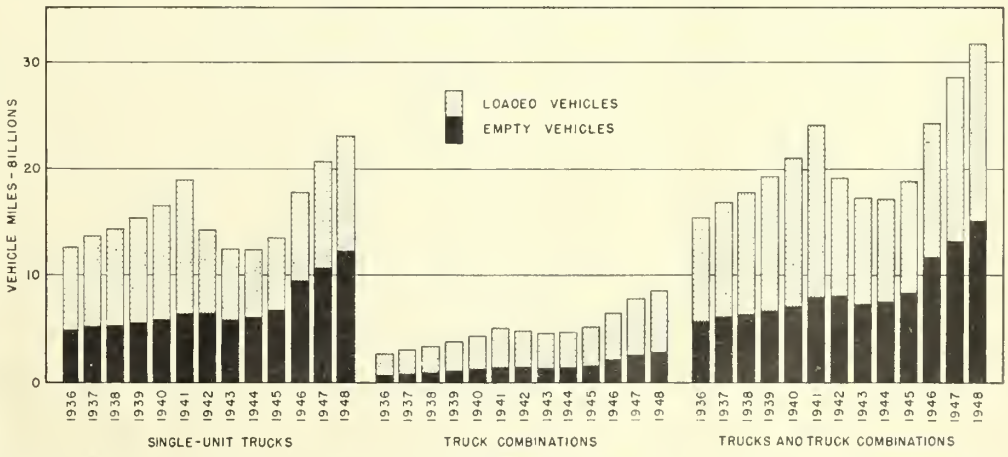


Figure 4.—Travel on main rural roads, 1936–48, by loaded and by empty trucks and truck combinations.

that of the preceding years show that increases for trucks and combinations were much greater than for passenger cars in all cases; and that increases for truck combinations were greater

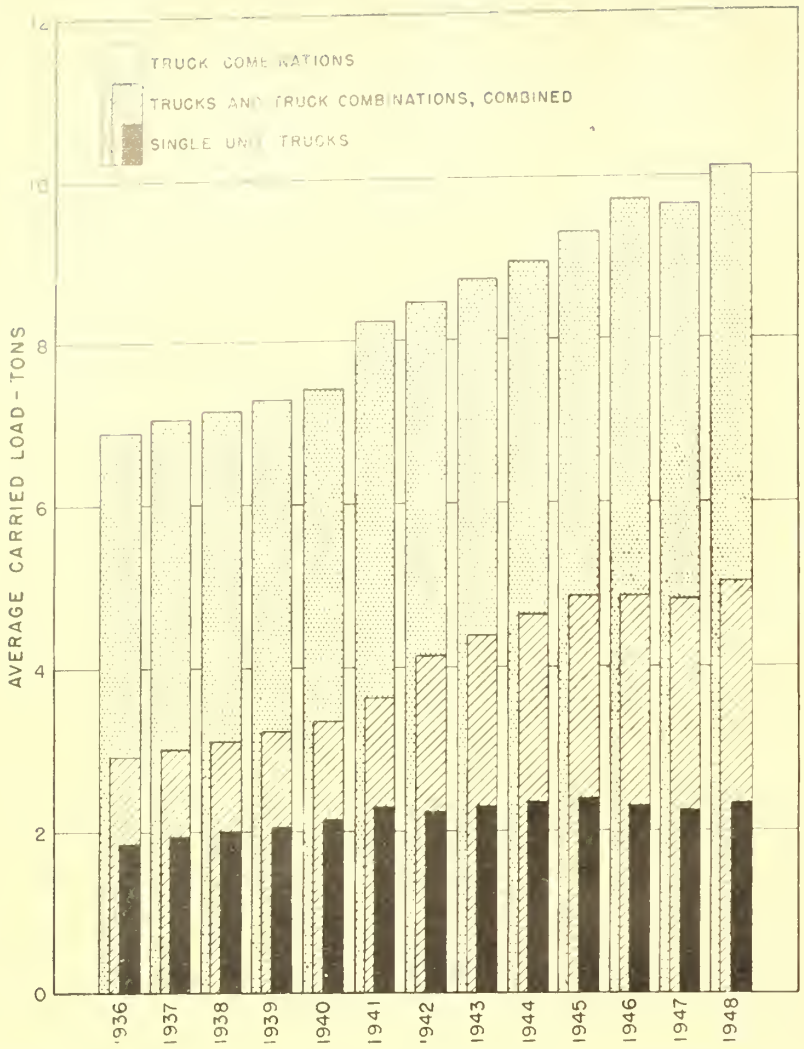


Figure 5.—Average load carried by trucks and truck combinations on main rural roads, 1936–48.

than for single-unit trucks except from 1947 to 1948. In the 12 years from 1936 to 1948, passenger-car travel increased about 60 percent, travel by all trucks and combinations doubled, and travel by truck combinations, considered separately, more than tripled.

Figure 5 gives a comparison of the average load carried by single-unit trucks, truck combinations, and all of these vehicles combined, in the 13 years that the planning surveys have been operating. The general trend of load weights was upward throughout the period. The slight decline in the weight of the load carried by single-unit trucks since 1945 has been more than offset by the increased use of combinations and the increased weight of the load of vehicles of this type.

Figure 6 shows a comparison for each year from 1936 through 1948 of the ton-miles of freight carried by trucks and truck combinations on main rural roads. The chart demonstrates clearly how truck combinations are transporting each year a larger portion of the total amount of highway freight. In 1936 the truck combinations hauled slightly less ton-mileage than the single-unit trucks, while in 1948 they hauled more than twice as much. The sharp increase in the total ton-mileage during the last 3 years is especially striking.

Percentage Loaded Remains Constant

In table 6 is shown a comparison of the percentage of vehicles carrying loads, the average carried load, and the ton-mileage carried for all trucks and combinations, for single-unit trucks, and for truck combinations in 1948 and the other significant periods used in table 5. The trend in average weight carried, shown graphically in figure 5, and that of the ton-mileage transported, shown in figure 6, have already been discussed. For the country as a whole, from 1947 to 1948, the percentage of trucks and truck combinations carrying loads decreased slightly. This decrease was not sufficient to offset a slight increase from 1946 to 1947, so the percentage loaded still remained slightly above that in 1946. It appears, then, that the downward trend in percentage loaded, noted each year from 1942 to 1946, inclusive, was halted in 1946, probably due to the fact that passenger cars were becoming more plentiful, and the tendency to use trucks in lieu of passenger cars began to decrease. Since 1946 the trend has not been clearly established. It was still true in 1948 that over half of the single-unit trucks passing the stations were empty, the percentage loaded being 47 percent in 1948 compared to 61 percent in 1936.

Table 7 gives a detailed comparison of the percentage of vehicle-miles of travel, percentage of vehicles loaded, average carried load, and percentage of total ton-miles carried by the various types of trucks and truck combinations traveling on main rural roads in 1947 and 1948. Many interesting comparisons can be made from this table, showing the relative importance from a freight-carrying standpoint of different portions of the traffic stream. For instance, it may be seen from the table that in 1948, while panel and pick-up trucks traveled almost 29 percent of the vehicle-miles, they accounted for less than 3 percent of the ton-mileage; likewise, while the tractor-truck and semi-trailer combinations

Table 6.—*Comparison of the estimated percentage of trucks and truck combinations loaded, average carried load, and ton-miles carried on main rural roads in 1936, 1941, 1946, 1947, and 1948*

Year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1936	62.8	2.90	28,005	60.7	1.86	14,258	72.2	6.90	13,747
1941	66.7	3.64	58,737	65.4	2.29	28,487	71.6	8.23	30,250
1941:1936 ratio	*1.06*	*1.26*	*2.10*	*1.08*	*1.23*	*2.00*	*.99*	*1.19*	*2.20*
1946	51.7	4.84	60,892	46.4	2.31	19,101	66.2	9.70	41,791
1946:1941 ratio	*.78*	*1.33*	*1.04*	*.71*	*1.01*	*.67*	*.92*	*1.18*	*1.38*
1946:1936 ratio	*.82*	*1.67*	*2.17*	*.76*	*1.24*	*1.34*	*.92*	*1.41*	*3.04*
1947	53.5	4.81	73,610	48.3	2.26	22,610	67.1	9.63	51,000
1948	52.2	5.02	83,119	46.8	2.33	25,219	66.5	10.10	57,900
1948:1947 ratio	*.98*	*1.04*	*1.13*	*.97*	*1.03*	*1.12*	*.99*	*1.05*	*1.14*
1948:1941 ratio	*.78*	*1.38*	*1.42*	*.72*	*1.02*	*.89*	*.93*	*1.23*	*1.91*
1948:1936 ratio	*.85*	*1.73*	*2.97*	*.77*	*1.25*	*1.77*	*.92*	*1.46*	*4.21*

traveled about 25 percent of the vehicle-mileage, they carried 63 percent of the ton-mileage.

From the portion of table 7 showing the percentage of vehicles carrying loads, by types, it can be observed that the percentage of vehicles carrying loads increases directly as the size of the vehicle type, extending from the light panel and pick-up trucks that are loaded 37 percent of the time to the heavy truck and trailer combinations that are loaded 70 percent of the time.

Frequency of Loads Above Legal Limit Increases

Table 8 shows the frequency of illegal loadings, expressed as the number of trucks and truck combinations of each type per 1,000 such vehicles counted, empties included, that exceeded the permissible axle, axle-group, or gross-weight legal limits in effect in the individual States in the summer of 1948. Violations of the State laws were most frequent in the East North Central region, where it was found that 84 of each 1,000 vehicles exceeded a State weight limit, 20 of these vehicles exceeding the limit by more than 20 percent. The Middle Atlantic region stood second in frequency of violations, for in this area 65 vehicles out of each 1,000 exceeded a load limit, and 17 such vehicles exceeded the limit by more than 20 percent.

A comparison of the frequency data concerning violations of State weight laws in 1948, as shown in table 8, with similar data collected in the previous year, indicates that these violations have increased in all areas except the Pacific and New England regions. In the Pacific region the frequency of violations decreased almost 25 percent, while in the New England region the frequency remained the same as in the previous year, at 35 per 1,000.

Comparison with Recommended Weight Limits

Uniform regulations concerning maximum allowable gross weights, axle weights, and axle-group weights have been adopted as a policy by the American Association of State Highway Officials and recommended to the various

(Text continued on page 298: tables 8-11 are on pages 294-5)

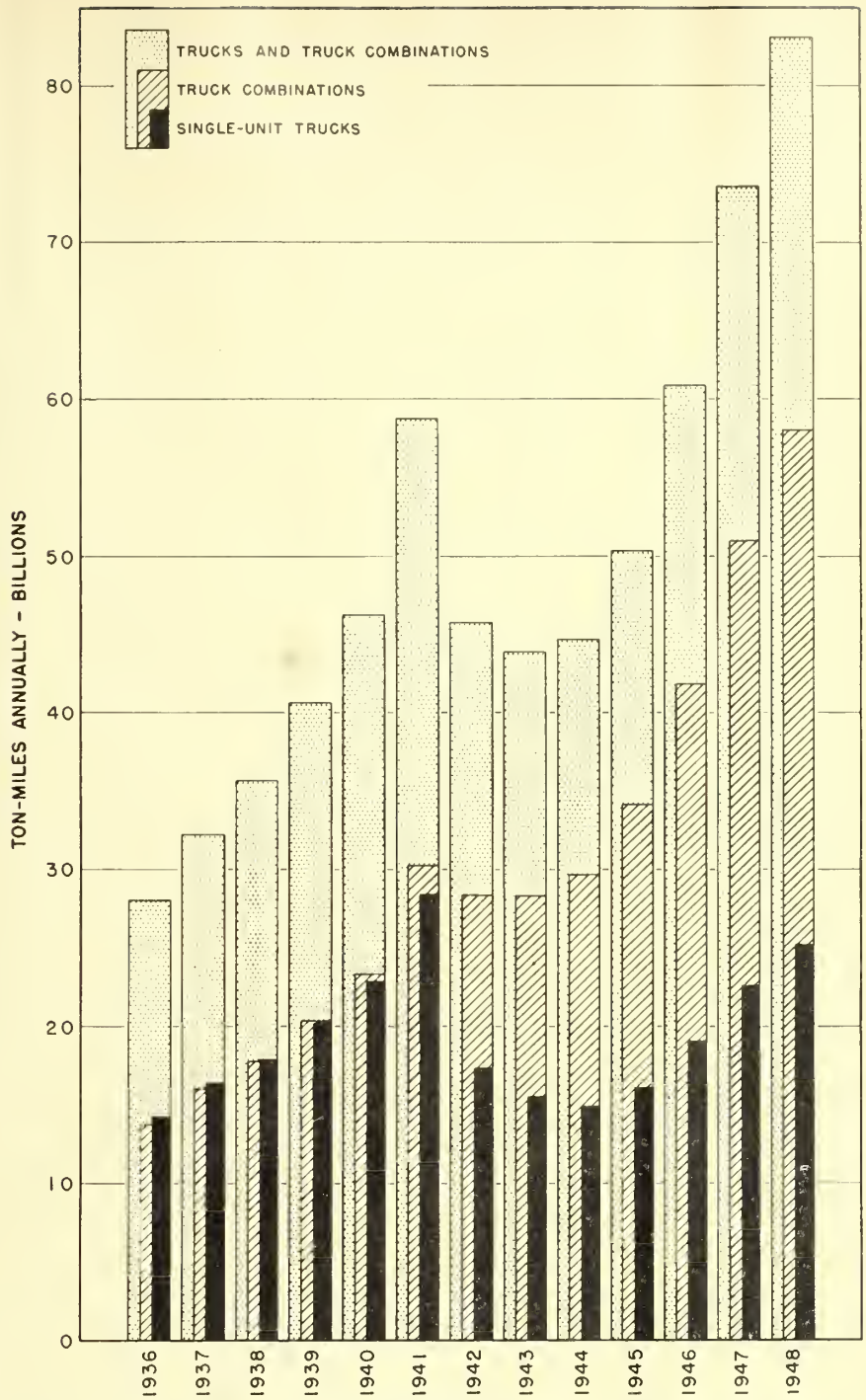


Figure 6.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936–48.

Table 7.—*Percentage of vehicle-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried by various types of trucks and truck combinations on main rural roads in 1948 compared to that in corresponding months of 1947*

Vehicle type	Percentage of vehicle-miles of travel		Percentage loaded		Average carried load		Percentage of ton-mileage carried	
	1948	1947	1948	1947	1948	1947	1948	1947
Single-unit trucks:					*Tons*	*Tons*		
Panel and pick-up	28.90	28.44	36.7	39.2	0.64	0.63	2.60	2.74
Other 2-axle, 4-tire	3.59	4.12	51.9	50.9	1.24	1.29	.88	1.06
Other 2-axle, 6-tire	38.66	36.18	53.6	54.4	3.10	2.91	24.51	22.26
3-axle	1.70	3.72	55.2	55.5	6.57	5.80	2.35	4.66
All single-unit trucks	72.85	72.46	46.8	48.3	2.33	2.26	30.34	30.72
Truck combinations:								
Tractor-truck and semitrailer	25.35	24.96	66.2	67.2	9.83	9.25	63.09	60.32
Truck and trailer	1.80	2.58	70.2	66.8	13.64	13.38	6.57	8.96
All truck combinations	27.15	27.54	66.5	67.1	10.10	9.63	69.66	69.28
All trucks and truck-combinations	100.00	100.00	52.2	53.5	5.02	4.81	100.00	100.00

Table 8.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle, axle-group, or gross-weight laws in effect in the States by various percentages (maximum) of overload, in the summer of 1948*

Region and type of vehicle (panel and pick-up trucks excluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than —				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	18	14	8	3	1	(1)
2-axle, 6-tire	97	69	55	27	3	--
3-axle	11	9	5	2	1	(1)
Average, single-unit trucks	124	90	62	30	16	2
Tractor-truck and semitrailer						
Truck and trailer	124	90	62	30	16	2
Average, truck combinations	35	26	17	8	4	(1)
Average, all trucks and combinations						
Middle Atlantic:						
2-axle, 4-tire	35	28	20	6	4	--
2-axle, 6-tire	133	106	86	56	56	12
3-axle	26	20	15	5	4	(1)
Average, single-unit trucks	186	127	101	55	27	4
Tractor-truck and semitrailer						
Truck and trailer	185	126	101	55	27	4
Average, truck combinations	65	46	36	17	10	1
Average, all trucks and combinations						
South Atlantic:						
2-axle, 4-tire	9	7	6	2	1	--
2-axle, 6-tire	105	71	51	26	9	--
3-axle	7	5	4	2	1	--
Average, single-unit trucks	190	126	84	34	15	1
Tractor-truck and semitrailer						
Truck and trailer	190	126	84	34	15	1
Average, truck combinations	56	37	25	11	5	(1)
Average, all trucks and combinations						
East North Central:						
2-axle, 4-tire	10	8	4	1	(1)	--
2-axle, 6-tire	62	32	32	26	19	--
3-axle	7	5	3	1	(1)	--
Average, single-unit trucks	194	141	95	47	19	3
Tractor-truck and semitrailer	673	565	331	147	103	13
Truck and trailer	212	157	104	51	22	3
Average, truck combinations	84	62	41	20	8	1
Average, all trucks and combinations						
East South Central:						
2-axle, 4-tire	23	15	9	3	(1)	--
2-axle, 6-tire	74	57	57	34	17	17
3-axle	14	9	5	2	(1)	(1)
Average, single-unit trucks	183	139	100	55	24	5
Tractor-truck and semitrailer						
Truck and trailer	183	139	100	55	24	5
Average, truck combinations	47	34	23	12	5	1
Average, all trucks and combinations						
West North Central:						
2-axle, 4-tire	1	--	--	--	--	--
2-axle, 6-tire	4	1	1	(1)	(1)	--
3-axle	6	4	--	--	--	--
Average, single-unit trucks	3	1	1	(1)	(1)	--
Tractor-truck and semitrailer	222	149	84	32	13	2
Truck and trailer	214	144	81	31	13	2
Average, truck combinations	50	33	19	7	3	(1)
Average, all trucks and combinations						
West South Central:						
2-axle, 4-tire	11	8	8	--	--	--
2-axle, 6-tire	9	6	3	1	(1)	--
3-axle	13	13	13	13	7	7
Average, single-unit trucks	5	3	2	1	(1)	(1)
Tractor-truck and semitrailer	166	127	89	44	22	6
Truck and trailer	49	49	49	49	49	--
Average, truck combinations	164	125	88	44	23	6
Average, all trucks and combinations	47	35	24	12	6	2
Mountain:						
2-axle, 4-tire	25	16	10	5	3	(1)
2-axle, 6-tire	8	8	4	4	--	--
3-axle	10	7	4	2	1	(1)
Average, single-unit trucks	174	143	102	46	20	4
Tractor-truck and semitrailer	178	112	75	44	27	5
Truck and trailer	175	138	98	46	21	4
Average, truck combinations	40	31	21	10	5	1
Average, all trucks and combinations						
Pacific:						
2-axle, 4-tire	12	7	3	1	(1)	--
2-axle, 6-tire	26	11	5	--	--	--
3-axle	8	4	2	(1)	(1)	--
Average, single-unit trucks	105	56	36	15	2	--
Tractor-truck and semitrailer	127	47	12	--	--	--
Truck and trailer	112	53	28	10	1	--
Average, truck combinations	45	21	11	4	(1)	--
Average, all trucks and combinations						
United States average:						
2-axle, 4-tire	1	1	1	--	--	--
2-axle, 6-tire	15	10	7	2	1	(1)
3-axle	61	42	33	20	15	3
Average, single-unit trucks	9	6	5	2	1	(1)
Tractor-truck and semitrailer	179	125	85	41	18	3
Truck and trailer	186	114	59	25	18	2
Average, truck combinations	179	124	83	40	18	3
Average, all trucks and combinations	55	38	26	12	6	1

[1] Less than 5 per 10,000.

Table 9.—*Number of axles, per 1,000 loaded and empty trucks and truck combinations that exceeded the permissible axle-load limit of 18,000 pounds recommended by the A. A. S. H. O. by various percentages of overload, in the summer of 1948*

Region and type of vehicle (panel and pick-up trucks exeluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than —				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	45	39	31	17	9	3
2-axle, 6-tire	95	62	39	17	--	--
3-axle	25	21	17	9	5	2
Average, siugle-unit trucks	432	353	278	147	65	10
Traetor-truck and semitrailer						
Truck and trailer	432	353	278	147	65	10
Average, truck combinations	112	92	73	38	18	4
Average, all trueks aud eombinations						
Middle Atlantic:						
2-axle, 4-tire	78	67	58	38	26	6
2-axle, 6-tire	208	148	146	65	63	3
3-axle	54	45	40	25	18	4
Average, siugle-unit trueks	672	583	499	313	152	41
Traetor-truck and semitrailer						
Truck and trailer	669	580	497	312	151	41
Average, truck eombinations	205	177	153	96	51	13
Average, all trucks and eombinations						
South Atlantic:						
2-axle, 4-tire	12	9	8	3	1	(1)
2-axle, 6-tire	96	64	64	6	--	--
3-axle	8	6	5	2	1	(1)
Average, single-unit trucks	346	249	170	73	26	2
Traetor-truck and semitrailer						
Truck and trailer	345	248	170	73	26	2
Average, truck eombinations	98	71	49	21	8	1
Average, all trucks and eombinations						
East North Ceutral:						
2-axle, 4-tire	11	8	4	1	(1)	--
2-axle, 6-tire	13	13	6	--	--	--
3-axle	7	5	2	1	(1)	--
Average, single-unit trucks	221	152	91	36	12	2
Traetor-truck and semitrailer	792	369	161	21	4	--
Truck and trailer	243	160	94	35	11	2
Average, truck eombinations	95	64	38	14	4	1
Average, all trucks and eombinations						
East South Central:						
2-axle, 4-tire	23	15	9	3	(1)	--
2-axle, 6-tire	58	40	40	17	--	--
3-axle	13	9	5	2	(1)	--
Average, single-unit trucks	236	174	120	56	23	4
Tractor-truck and semitrailer						
Truck and trailer	236	174	120	56	23	4
Average, truck eombinations	56	41	27	12	4	1
Average, all trucks and combinations						
West North Central:						
2-axle, 4-tire	(1)	--	--	--	--	--
2-axle, 6-tire	5	2	1	(1)	(1)	--
3-axle	3	--	--	--	--	--
Average, single-unit trucks	3	1	1	(1)	(1)	--
Tractor-truck and semitrailer	210	124	64	21	8	2
Truck and trailer	203	120	62	20	8	2
Average, truck eombinations	47	27	15	4	2	(1)
Average, all trucks and combinations						
West South Central:						
2-axle, 4-tire	11	8	8	--	--	--
2-axle, 6-tire	8	7	3	1	(1)	--
3-axle	20	13	13	13	13	--
Average, all single-unit trucks	9	3	2	1	(1)	--
Tractor-truck and semitrailer	197	144	93	45	21	6
Truek and trailer	137	127	88	--	--	--
Average, truck combinations	196	143	92	44	21	6
Average, all trucks and combinations	58	40	26	12	5	2
Mountain:						
2-axle, 4-tire	23	16	10	5	3	--
2-axle, 6-tire	4	4	4	4	--	--
3-axle	9	7	4	2	1	--
Average, all single-unit trucks	135	95	66	20	7	--
Tractor-truck and semitrailer	112	73	52	37	18	5
Truek and trailer	131	91	64	23	9	1
Average, truck eombinations	32	23	15	6	2	(1)
Average, all trucks and eombinations						
Pacifie:						
2-axle, 4-tire	12	7	4	(1)	(1)	--
2-axle, 6-tire	13	2	2	--	--	--
3-axle	7	4	2	(1)	(1)	--
Average, single-unit trucks	79	42	20	6	1	--
Tractor-truck and semitrailer	97	36	10	--	--	--
Truck and trailer	85	40	17	4	1	--
Average, truck eombinations	35	17	7	1	(1)	--
Average, all trucks and combinations						
United States average:						
2-axle, 4-tire	1	1	1	--	--	--
2-axle, 6-tire	22	18	14	8	5	1
3-axle	64	43	41	15	12	1
Average, single-unit trucks	14	11	9	5	3	1
Traetor-truck and semitrailer	281	211	150	76	33	7
Truck and trailer	179	82	36	6	3	1
Average, truek eombinations	274	202	142	71	31	6
Average, all trueks and combinations	85	63	45	23	11	2

[1] Less than 5 per 10,000.

Table 10.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle-group loads recommended by the A. A. S. H. O. by various percentages of overload, in the summer of 1948*

Region and type of vehicle (panel and pick-up trucks excluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than —				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	1	1	(1)	---	---	--
3-axle	84	74	55	17	3	--
Average, single-unit trucks	2	2	1	(1)	(1)	--
Tractor-truck and semitrailer	66	46	26	9	3	(1)
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	66	46	26	9	3	(1)
Average, all trucks and combinations	16	11	6	2	1	(1)
Middle Atlantic:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6 tire	5	1	---	---	---	--
3-axle	154	136	98	59	47	30
Average, single-unit trucks	8	5	3	2	2	1
Tractor- truck and semitrailer	201	150	112	60	37	8
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	200	149	111	60	37	8
Average, all trucks and combinations	55	40	30	16	11	3
South Atlantic:						
2-axle, 4-tire	---	---	---	---	---	--
2 axle, 6-tire	---	---	---	---	---	--
3-axle	83	58	36	21	9	--
Average, single- unit trucks	2	1	1	(1)	(1)	--
Tractor-truck and semitrailer	45	31	22	11	7	1
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	45	31	22	11	7	1
Average, all trucks and combinations	14	9	7	3	2	(1)
East North Central:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	---	---	---	---	---	--
3-axle	30	13	---	---	---	--
Average, single-unit trucks	1	(1)	---	---	---	--
Tractor-truck and semitrailer	103	76	55	26	11	2
Truck and trailer	763	684	633	361	205	68
Average, truck combinations	128	99	77	39	18	4
Average, all trucks and combinations	49	37	29	15	7	1
East South Central:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	---	---	---	---	---	--
3-axle	17	17	17	17	--	--
Average, single-unit trucks	(1)	(1)	(1)	(1)	---	--
Tractor- truck and semitrailer	34	19	12	7	3	--
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	34	19	12	7	3	--
Average, all trucks and combinations	7	4	2	1	1	--
West North Central:						
2-axle, 4-tire	---	---	---	---	---	--
2 axle, 6 tire	---	---	---	---	---	--
3-axle	12	12	---	---	---	--
Average, single-unit trucks	(1)	(1)	---	---	---	--
Tractor-truck and semitrailer	109	73	45	12	4	(1)
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	105	70	43	12	4	(1)
Average, all trucks and combinations	23	16	10	3	1	(1)
West South Central:						
2-axle, 4-tire	11	8	8	---	---	--
2-axle, 6-tire	---	---	---	---	---	--
3-axle	7	7	7	7	7	7
Average, single-unit trucks	(1)	(1)	(1)	(1)	(1)	(1)
Tractor-truck and semitrailer	41	33	26	14	6	2
Truck and trailer	49	49	49	49	39	39
Average, truck combinations	41	33	26	15	7	4
Average, all trucks and combinations	11	9	7	4	2	1
Mountain:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	---	---	---	---	---	--
3-axle	15	8	4	---	---	--
Average, single-unit trucks	(1)	(1)	(1)	---	---	--
Tractor-truck and semitrailer	119	88	62	24	13	(1)
Truck and trailer	223	143	104	36	11	5
Average, truck combinations	136	97	69	26	13	1
Average, all trucks and combinations	25	18	13	5	2	(1)
Pacific:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	---	---	---	---	---	--
3-axle	11	2	2	---	---	--
Average, single-unit trucks	1	(1)	(1)	---	---	--
Tractor-truck and semitrailer	148	105	67	22	11	--
Truck and trailer	233	149	80	11	2	--
Average, truck combinations	175	119	71	19	8	--
Average, all trucks and combinations	63	42	25	7	3	--
United States average:						
2-axle, 4-tire	(1)	(1)	(1)	---	--	--
2-axle, 6-tire	(1)	(1)	(1)	---	---	--
3-axle	53	41	27	15	11	6
Average, single-unit trucks	1	1	1	(1)	(1)	(1)
Tractor-truck and semitrailer	97	70	49	22	11	2
Truck and trailer	272	198	143	58	29	11
Average, truck combinations	109	78	55	24	12	3
Average, all trucks and combinations	30	22	16	7	3	1

[1] Less than 5 per 10,000.

Table 11.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible motor-vehicle loads recommended by the A. A. S. H. O. by various percentages (maximum) of overload, summer of 1948*

Region and type of vehicle (panel and pick-up trucks excluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than —				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4 tire	---	---	---	---	---	--
2-axle, 6-tire	45	39	30	16	8	2
3-axle	99	77	59	17	3	--
Average, single-unit trucks	25	21	17	8	4	1
Tractor-truck and semitrailer	285	240	196	109	56	8
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	285	240	196	109	56	8
Average, all trucks and combinations	80	68	55	30	15	2
Middle Atlantic:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	78	67	58	38	26	6
3-axle	169	159	139	59	47	30
Average, single-unit trucks	53	46	40	25	17	5
Tractor-truck and semitrailer	433	404	362	256	138	42
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	431	402	360	255	137	42
Average, all trucks and combinations	146	134	119	82	47	14
South Atlantic:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	12	9	8	3	1	(1)
3-axle	108	84	58	27	11	2
Average, single-unit trucks	8	6	5	2	1	(1)
Tractor-truck and semitrailer	245	184	128	59	25	3
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	244	184	128	59	25	3
Average, all trucks and combinations	71	54	38	17	7	1
East North Central:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	10	8	4	1	(1)	--
3-axle	30	13	13	---	---	--
Average, single-unit trucks	7	4	3	1	(1)	--
Tractor-truck and semitrailer	196	149	100	44	17	4
Truck and trailer	821	739	637	400	209	68
Average, truck combinations	220	171	120	57	24	6
Average, all trucks and combinations	87	67	47	22	9	2
East South Central:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	23	15	9	3	(1)	--
3-axle	40	40	40	17	---	--
Average, single-unit trucks	13	9	5	2	(1)	--
Tractor-truck and semitrailer	172	131	91	46	20	4
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	172	131	91	46	20	4
Average, all trucks and combinations	44	33	22	11	4	1
West North Central:						
2-axle, 4-tire	1	---	---	---	---	--
2-axle, 6-tire	5	2	1	(1)	(1)	(1)
3-axle	15	15	---	---	---	--
Average, single-unit trucks	3	1	1	(1)	(1)	(1)
Tractor-truck and semitrailer	209	140	82	27	10	2
Truck and trailer	---	---	---	---	---	--
Average, truck combinations	202	135	79	26	10	2
Average, all trucks and combinations	47	31	18	6	2	(1)
West South Central:						
2-axle, 4-tire	11	8	8	---	---	--
2-axle, 6-tire	8	6	3	1	---	--
3-axle	13	13	13	13	7	7
Average, single-unit trucks	4	3	2	1	(1)	(1)
Tractor-truck and semitrailer	149	115	78	40	16	4
Truck and trailer	49	49	49	49	39	39
Average, truck combinations	147	114	77	40	16	5
Average, all trucks and combinations	41	32	22	11	4	1
Mountain:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	23	15	10	5	2	--
3-axle	15	8	4	4	---	--
Average, single-unit trucks	10	6	4	2	1	--
Tractor-truck and semitrailer	163	129	94	42	18	3
Truck and trailer	232	157	110	48	25	5
Average, truck combinations	174	134	97	43	19	3
Average, all trucks and combinations	40	30	21	10	4	1
Pacific:						
2-axle, 4-tire	---	---	---	---	---	--
2-axle, 6-tire	11	7	3	1	(1)	--
3-axle	19	5	5	---	---	--
Average, single-unit trucks	7	4	2	(1)	(1)	--
Tractor-truck and semitrailer	181	126	78	29	11	--
Truck and trailer	320	182	87	11	2	--
Average, truck combinations	225	144	81	23	8	--
Average, all trucks and combinations	84	54	30	8	3	--
United States average:						
2-axle, 4-tire	1	1	1	---	---	--
2-axle, 6-tire	22	17	14	8	5	1
3-axle	63	51	41	17	11	6
Average, single-unit trucks	13	10	8	5	3	1
Tractor-truck and semitrailer	226	178	131	70	33	8
Truck and trailer	336	228	149	64	31	11
Average, truck combinations	233	181	132	69	32	7
Average, all trucks and combinations	73	56	42	23	11	3

[1] Less than 5 per 10,000.

Trends in Motor-Vehicle Travel, 1948

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by G. P. St. CLAIR, Chief of Branch

Total motor-vehicle travel in 1948 is estimated as more than 397 billion vehicle-miles, about evenly divided between rural highways and urban roads and streets. Passenger cars accounted for 319 billion vehicle-miles of the total, busses 4 billion, and trucks and combinations 74 billion. Except for urban travel of busses, estimated travel in 1948 was greater than in 1947 for every category, although in all cases the increases were smaller than those from 1946 to 1947. As in the previous year, truck travel in 1948 increased at about twice the rate of passenger-vehicle travel.

The average vehicle in 1948 traveled 9,707 miles, using 741 gallons of motor fuel at a rate of 13 miles per gallon. Annual average passenger-car travel continued to decrease from the 1946 peak, while average truck-mileage, though maintaining a steady rise, was still below the 1941 maximum.

Total travel has been very nearly equally divided between rural highways and urban roads and streets since the end of the war. As indicated in this article, the ratios of rural to urban travel for the past 3 years vary in a range of less than 2 percent.

THIS article is the fourth of an annual series in PUBLIC ROADS [1] giving classified estimates of motor-vehicle travel in the United States. The procedures used in making these estimates were the same as those described in the previous reports, the principal factors controlling the calculations being (1) the annual estimates of rural-road traffic described in another article in this issue of PUBLIC ROADS; (2) the annual reports of highway use of motor fuel; and (3) reported motor-vehicle registrations. As in the previous calculations, the 1948 estimates were based on indicated changes from a previous year, the year 1947 being used as the base year in this instance.

Table 1 reports, for the various classes of motor vehicles, the estimates for 1948 of rural, urban, and total vehicle-miles traveled, average miles traveled per vehicle, motor-fuel consumption, in total and per vehicle, and average travel per gallon of motor fuel consumed. The numbers of registered motor vehicles, as modified for the purpose of these estimates, are also given.

The total travel of motor vehicles in 1948 is estimated as 397,589 million vehicle-miles, of which 198,507 million were traveled on rural highways, and 199,082 million on urban highways and streets. The total travel of passenger cars was 319,459 million vehicle-miles, of which the travel on rural roads was 151,275 million and the urban travel 168,184 million. The estimated travel of trucks and combinations, 73,847 million vehicle-miles, included 45,096 million traveled on rural roads, and 28,751 million on urban roads and streets.

[1] *Trends in motor-vehicle travel, 1936 to 1945*, PUBLIC ROADS, vol. 24, No. 10, Oct.-Nov.-Dec. 1946; *Trends in motor-vehicle travel, 1946*, PUBLIC ROADS, vol. 25, No. 3, March 1948; *Trends in motor-vehicle travel, 1947*, PUBLIC ROADS, vol. 25, No. 7, March 1949.

Percentage Changes in Annual Travel

Table 2 gives, for the major vehicle types, a comparison of the percentage changes in 1948 travel over that of 1947, with the corresponding changes in 1947 travel over that of 1946. Estimated travel in 1948 was greater than in 1947 in all categories except the urban travel of busses, which suffered a slight loss. The percentage increases over 1947, however, were somewhat less in most cases than those of 1947 over 1946. The increase in passenger-car travel was 6.39 percent, in contrast to a 1947 increase of 7.07 percent. Travel of trucks and combinations increased by 11.74 percent, as against an increase of 17.71 percent in 1947. The increase in total travel was reduced from 8.80 percent in 1947 to 7.28 in 1948.

The increase in the estimated travel of trucks and combinations, 11.74 percent, was nearly twice that found in the case of passenger cars. This finding, which continues a trend displayed in the 1947 estimates, is attributable in part to the fact that the numbers of truck registrations have been increasing more rapidly than those of passenger cars. In 1948 the registrations of trucks and combinations were 11 percent greater than in 1947, whereas passenger-car registrations increased only 8 percent.

The estimates of urban travel of both passenger cars and trucks show a greater increase

Table 1.— *Classified estimate of motor-vehicle travel in the United States in the calendar year 1948*

Vehicle type	Motor-vehicle travel			Number of registered vehicles [1]	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Rural travel	Urban travel	Total travel			Total [2]	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles*
Passenger vehicles:								
Passenger cars [3]	151,275	168,184	319,459	33,394	9,566	21,369	640	14.95
Busses:								
Commercial	1,474	2,074	3,548	92	38,500	709	7,700	5.00
School and nonrevenue	662	73	735	92	8,000	71	769	10.40
All busses	2,136	2,147	4,283	184	23,268	780	4,239	5.49
All passenger vehicles	153,411	170,331	323,742	33,578	9,642	22,149	660	14.62
Trucks and combinations	45,096	28,751	73,847	7,379	10,008	8,189	1,110	9.02
All motor vehicles	198,507	199,082	397,589	40,957	9,707	30,338	741	13.11

[1] These registration totals differ from those given in Bureau of Public Roads table MV-1 for 1948 because of the following adjustments: (1) Approximate correction for defective classification in 3 States, as described in footnotes 9, 10, and 13 of that table; (2) inclusion of publicly owned vehicles, listed separately in table MV-1; (3) reduction of private and commercial truck registrations by 2.5 percent to allow for registrations in more than 1 State; and (4) substitution of bus totals as estimated by the bus industry to afford a complete segregation of commercial busses from school and nonrevenue busses and to allow for registrations in more than 1 State.

[2] Total highway use of motor fuel in 1948 is given as 30,461 million gallons in Bureau of Public Roads table G-21. For this analysis there was deducted from that total 123 million gallons estimated use by motorcycles (250 gallons per motorcycle).

[3] Including taxicabs.

Table 2.—*Percentage changes in motor-vehicle travel, 1948 over 1947, compared with corresponding changes, 1947 over 1946*

Vehicle type	Percentage change in travel—					
	1948 over 1947			1947 over 1946		
	Rural	Urban	Total	Rural	Urban	Total
Passenger cars	5.30	7.38	6.39	7.20	6.95	7.07
Busses	1.67	−.14	.75	5.10	4.72	4.91
All passenger vehicles	5.25	7.28	6.31	7.17	6.92	7.04
Trucks and combinations	10.61	13.56	11.74	17.86	17.47	17.71
All motor vehicles	6.42	8.15	7.28	9.34	8.26	8.80

over 1947 values than do those of travel on rural roads. This finding is a reversal of the relationships displayed by the 1947 estimates.

Table 3 gives values of the estimated rural, urban, and total travel volumes in the years 1941 through 1948. With respect to total travel, there are also given values of the ratio of the volume in each year to the 1941 volume, and the ratio to the volume in the preceding year. It will be observed that, after a recovery in 1946 to a value slightly above that of 1941, there has been a steady rise in the volume of total travel to a level in 1948 that was 19 percent above that of 1941. The travel estimates of recent years are indicative of a firm upward trend, with some tendency for the values to round over toward a position of stability which would appear to be far in the future.

Average Annual Mileages

Figure 1 gives a comparison of trends in the average miles per vehicle traveled by passenger cars and by trucks and combinations in the years 1936 to 1948, inclusive. It will be noted that the average mileage of passenger cars, after recovering from the wartime decline to a maximum of 9,942 miles in 1946, declined in 1947 to 9,727 miles, and again in 1948 to 9,566 miles. The average mileage of trucks and combinations, on the other hand, has been rising steadily from its low value in 1944, although the 1948 value is still well below that of 1941.

In the discussion of the 1947 estimates,[1] the observed decrease in the average annual mileage of passenger cars was attributed to the fact that the acute postwar shortage of automobiles caused those in service to be driven greater annual mileages than would have been the case had passenger cars been in full supply. As production increased and the demand for new cars was partly satisfied, there was a tendency for the average mileage driven to diminish. It was stated that "It is not unlikely that the trend toward somewhat decreased average annual mileage of passenger cars will continue for a year or two as a result of the greatly increased production of these vehicles." The 1948 estimate of 9,566 miles per passenger car tends to confirm this prediction.

The average mileages of trucks show a more normal trend than those of passenger cars, probably because the shortage in these vehicles was not as acute in relation to the demand as in the case of passenger cars. The failure of the average mileages of trucks and combinations to reach their prewar levels may reasonably be attributed to the fact, previously cited, that there has been an enormous increase in truck registrations during the postwar years, with the result that the large volume of increased trucking business can be carried at an average mileage per vehicle lower than that of the prewar years. Truck registrations in 1948 were more than 44 percent above those of 1941. Passenger-car registrations, on the other hand, were only about 12½ percent above their 1941 level.

As would be expected from the showing of passenger cars and trucks, the average annual mileages for all motor vehicles combined show a moderate decline, the successive values being 9,958 in 1946, 9,831 in 1947, and 9,707 in 1948.

[1] *Trends in motor-vehicle travel, 1947*, Public Roads, vol. 25, No. 7, March 1949, p. 157.

Table 3.—*Comparison of estimates of rural, urban, and total travel in the years 1941–48*

Year	Rural travel	Urban travel	Ratio of rural to urban travel	Total travel		
				Amount	Ratio to 1941 total	Ratio to total in preceding year
	Million vehicle-miles	*Million vehicle-miles*		*Million vehicle-miles*		
1941	169,805	163,591	1.0380	333,396	1.0000	[1] 1.1034
1942	128,861	138,235	.9322	267,096	.8011	.8011
1943	97,757	108,990	.8969	206,747	.6201	.7741
1944	100,830	110,750	.9104	211,580	.6346	1.0234
1945	119,183	129,743	.9186	248,926	.7466	1.1765
1946	170,606	170,049	1.0033	340,655	1.0218	1.3685
1947	186,534	184,088	1.0133	370,622	1.1117	1.0880
1948	198,507	199,082	.9971	397,589	1.1925	1.0728

[1] The estimated total travel in 1940 was 302,143 million vehicle-miles.

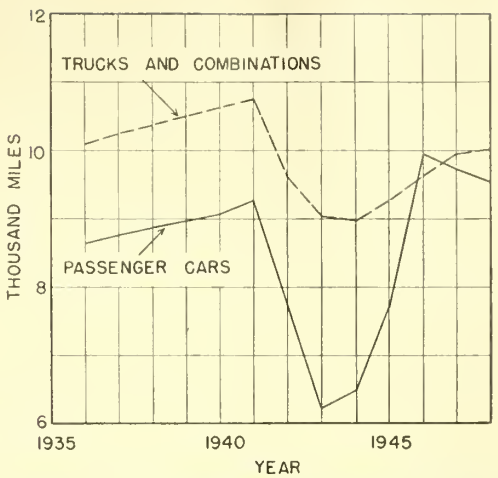


Figure 1.—Comparison of trends in the annual average miles per vehicle traveled by passenger cars and by trucks and combinations.

Relative Changes in Rural and Urban Travel

The fourth column of table 3 gives, for each year beginning with 1941, the ratio of total rural travel to total urban travel. In 1941, the volume of rural travel exceeded that of urban travel by 3.8 percent. The heavy restrictions on the use of vehicles and gasoline during the war period caused this ratio to decline to a value slightly under 0.9 in 1943. In 1946, the ratio of rural to urban travel crossed the line again at the value 1.0033. The 1947 value of 1.0133 seemed to indicate that the trend would be toward recovery of the prewar position, where the volume of rural travel was definitely greater than that of urban travel, although by only a small percentage. The 1948 estimates have resulted in a reduction of the ratio to the value 0.9971. In comparing these ratios, it will be noted that the 1946, 1947, and 1948 values vary within a range of less than 2 percent.

Pertinent factors

In table 4 there are given values for the years 1947 and 1948 of various pertinent factors which go far to explain why the vehicle-mile estimates caused the estimated urban travel to be slightly greater than that of rural travel in 1948. In the first two lines of the table, it is shown that, whereas the estimated travel on rural roads in 1948 was only 6.42 percent above the 1947 value, the highway use of motor fuel, as given by Public Roads table G–21, 1948, exceeded the 1947 value by 7.96 percent. In other words, motor-fuel use increased more rapidly in 1948 than did rural-road traffic. It is natural to conclude, therefore, that, unless there was some drastic change in miles-per-gallon relationships, urban travel must have increased in greater proportion than rural travel in order to absorb the increased use of gasoline.

It should be noted that the various factors listed in table 4 are independent of the procedures used in making the estimates of total travel and urban travel. They are derived from the rural traffic estimates, which are accepted unchanged, and from the annual

Table 1.—*Comparison, for the years 1947 and 1948, of various factors relating to travel on rural roads, motor-fuel consumption, and motor-vehicle registrations* [1]

Item		1947	1948	Percentage, 1948 of 1947
For all vehicles:				
Highway use of motor fuel	thousand gallons	28,215,705	30,460,641	107.96
Rural-road travel	million vehicle-miles	186,534	198,507	106.42
Motor-vehicle registrations [2]		37,841,498	41,151,326	108.75
Rural-road travel per registered vehicle	miles	4,929	4,824	97.87
Motor-fuel consumption per registered vehicle	gallons	745.6	740.2	99.28
For passenger vehicles:				
Rural-road travel	million vehicle-miles	145,763	153,411	105.25
Passenger-vehicle registrations [3]		31,032,807	33,595,996	108.26
Rural-road travel per vehicle		4,697	4,566	97.21
For trucks and combinations:				
Rural-road travel	million vehicle-miles	40,771	45,096	110.61
Truck and tractor-truck registrations [3]		6,808,691	7,555,330	110.97
Rural-road travel per vehicle		5,988	5,969	99.68

[1] None of the values given in this table are dependent on approximations used in making the vehicle-mile estimates. They are derived from the rural-road travel estimates (based largely on automatic recorder data), and from the annual statistical tables published by the Bureau of Public Roads.

[2] From Bureau of Public Roads table G-21. [3] From Bureau of Public Roads tables MV-1 and MV-7.

motor-vehicle registration and gasoline-consumption tables.

Table 4 also gives the numbers of motor-vehicle registrations in 1947 and 1948, and by the use of these figures indicates the amount of rural-road traffic in miles per registered motor vehicle, and the motor-fuel consumption in gallons per registered vehicle. Similar calculations are carried out for the rural-road traffic of passenger vehicles and of trucks and combinations. Although not all motor vehicles appear on rural roads, the use of the total registration figures give highly indicative averages. It will be observed that the rural-road travel per registered motor vehicle was less in 1948 than in 1947, the 1948 value being 97.87 percent of the 1947 value. Motor-fuel consumption per registered vehicle was very slightly less in 1948 than in 1947, the percentage being 99.28. These figures tend to confirm the estimate of a decreased annual mileage of motor vehicles in 1948, but they also show why the estimates of urban travel had to indicate a relatively greater increase in 1948 than the estimates of rural travel. It was, in effect, a question of absorbing gasoline consumption not accounted for by rural travel.

Similar indications are given by the calculations with respect to passenger vehicles and trucks and combinations in the lower portion of table 4. The rural-road travel of passenger vehicles in terms of miles per vehicle was in 1948 only 97.21 percent of its 1947 value. In the case of trucks and combinations the 1947 and 1948 values are much closer, the percentage being 99.68. Comparison of the passenger-car and truck figures tends to confirm the finding that the travel of trucks and combinations is increasing at a more rapid rate than the travel of passenger cars.

More data needed

The data given in table 4 have been included to make it plain that the variation from year to year in the ratios of rural and urban traffic volumes results from the findings with respect to rural-road traffic volumes, reported motor-fuel consumption, and reported motor-vehicle registrations. The difficulty experienced in this connection is illustrative of the fact that a more comprehensive accumulation of data is needed in order to increase the accuracy of annual estimates of motor-vehicle travel.

(*Continued from page 293*)

State governments for adoption.[5] This policy recommends that no axle shall carry a load in excess of 18,000 pounds and no group of axles shall carry a load in excess of amounts specified in a table of permissible weights based on the distance between the extremes of any group of axles.

In table 9 is shown the number of axles per 1,000 vehicles of various types that exceeded the axle-load limit of 18,000 pounds recommended by the A.A.S.H.O. and the number exceeding these limits by various percentages. This table emphasizes again the high frequency of heavy axle loads in the Middle Atlantic and New England regions. The number of axles, per 1,000 vehicles, weighing more than the recommended limits was 32 in the Mountain, 35 in the Pacific, 205 in the Middle Atlantic, and 112 in the New England region. For truck combinations, corresponding figures are: Mountain 131, Pacific 85, Middle Atlantic 669, and New England 432.

Eliminating the empty vehicles (which are included in table 9), there were in the Middle Atlantic region almost six axle loads in excess of the recommended limit for each five loaded truck combinations, while in the New England region the corresponding figures were about three excessive axle loads for each five truck combinations.

In considering the data concerning frequency of axles carrying over 18,000 pounds in the Middle Atlantic and New England regions, the fact should be recognized that higher limits generally are permissible in this area and therefore the axles exceeding the recommended limits by 25 percent may be within the legal limits of certain States in these two regions. Comparison of the frequency data given in table 9 with those in table 8 shows that, in these two regions, of each three vehicles exceeding the recommended axle limit, one vehicle actually exceeded a State legal limit.

For the United States as a whole, the frequency of 18,000-pound or heavier axle loads of all trucks and truck combinations in 1948 was almost 29 percent higher than in 1947, while the frequency of those axle loads that exceeded the recommendation by 20 percent or more was over 50 percent higher.

Table 10 shows the number of vehicles per 1,000 vehicles of various types with an axle-group load in excess of the limits recommended by the American Association of State Highway Officials, and in excess of these limits by various percentages. For the country as a whole, of each 1,000 loaded and empty trucks and truck combinations, 30 had axle-group loads weighing in excess of the recommended limits, 7 of which exceeded the limits by more than 20 percent. It was the truck combinations which produced most of the high load concentrations. Of each 1,000 combinations weighed in the United States as a whole, 109 had axle-group loads weighing more than the recommended limits, of which 24 exceeded the limits by more than 20 percent.

For the United States as a whole, the frequency of axle-group loads of all trucks and truck combinations in excess of the recommendations was 20 percent higher in 1948 than in the previous year, while loads 20 percent or more in excess of the recommendations were 40 percent higher.

As might be expected, many vehicles were so loaded that they exceeded more than one recommended weight limit; and some vehicles had more than one axle loaded in excess of the recommended limit. Counting each vehicle only once, regardless of the number of ways in which it exceeded any of the recommended limits, table 11 was derived showing the number of vehicles per 1,000 (both loaded and empty included) of each type that exceeded the limits by various percentages. Those vehicles which exceeded more than one provision of the recommended restrictions were tabulated in the column showing the highest percentage excess of any item.

In the United States as a whole, 73 vehicles out of every 1,000 were overloaded to some degree, according to American Association of State Highway Officials standards, and 23 out of every 1,000 exceeded some one of the provisions by more than 20 percent. The frequency of vehicles exceeding the recommendations by any amount in 1948 was almost 24 percent higher than in the previous year, and the frequency of those that exceeded the recommendations by more than 20 percent was over 53 percent higher.

[5] *Policy concerning maximum dimensions, weights, and speeds of motor vehicles to be operated over the highways of the United States*, adopted April 1, 1946, by the American Association of State Highway Officials; published by the Association in 1946.

U. S. GOVERNMENT PRINTING OFFICE: 1950

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